Vol. L. No. 5

TABLE OF CONTENTS.

(24 pages; no inset.)

	Page.
ENGINEERING NEWS OF THE WEEK	89
LEADING ARTICLES:	
Sixteen-Story Concrete-Steel Office Building at Cincinnati, O. (illustrated)	90
Prospecting and Valuing Dredging Ground with a Well-Driller at Oroville, Cal. Newton Booth Knox.	94
Rules of Award for the John Fritz Medal	96
The Strength of Concrete as Affected by Different Percentages of Water. T. L. Doyle and E. R. Justice.	97
Tests of Portland Cement Mortar Exposed to Cold. C. S. Gowen.	98
The Retirement of Rear Admiral Geo. W. Melville, Engineer in Chief, United States Navy. W. M. McFarland.	98
Electric Pillar Crane for Handling Cupola Charges (illustrated)	99
A Large Ore Dock at Escanaba, Mich.; Chicago & Northwestern Ry. (illustrated)	102
Springs and Spring Steel. Wm. Metcalf.	104
A Heavy-Service Lathe with Electric Drive	106
Rational Formulas for the Strength of a Concrete-Steel Beam (illustrated). S. E. Slocum.	107
Some Recommendations Concerning Electrical and Mechanical Specifications of Trolley Line Insulators (illustrated). Samuel Sheldon and John D. Kelly.	109
Rivets in Structural Steel Work (illustrated). C. J. Tilden.	110
The Longest Submarine Telephone Cable (illustrated)	110
A Graphical Method of Setting Slope Stakes (illustrated). J. M. Rudiger.	111
Effect of Fineness of Grain on the Feasibility of Clay. Heinrich Ries.	111
Air Heating with Radiators and a Closed Piping System (illustrated)	111
A Municipal Asphalt Plant Recommended for New York City	112
EDITORIAL COMMENT.—Newspaper Logic on Dam Building—Transportation of Garbage by Street Railways—Municipal Ownership of Public Utilities at Winnetka, Ill.	100
EDITORIAL.—Notes on Recent Cement Tests.	100
LETTERS TO THE EDITOR.—Medical Outfits for Camping Parties: H. M. Wilson—A Heavy Rainfall at Baltimore, Md.: B. T. Fendall—The Heaviest Recorded Rainfall at Atlantic City, N. J.: Kenneth Allen—Correct Figures for Urban Transportation in New York City: M. A. C.—Prior Use of Parallelogram for Interpolating Contours (illustrated): Irving T. Farnham—A Union Engineering Building Held to be of No Benefit Unless the National Engineering Societies Combine: F. Stuart Williamson—Some Comparisons Between the Original and the New Designs for the Manhattan Bridge: G. Lindenthal—Criticisms of the Report of the Special Committee of Engineers on the Plans for the Manhattan Bridge: Gus C. Henning	101

A PECULIAR SYSTEM OF ELECTRIC TRACTION has been tried at Devonport, England. The tramway line there is about five miles long, and has numerous and heavy grades. This has been operated under the usual direct-current system, with series-wound motors and series-parallel control. For enabling the descending grades to be utilized to return power to the line, and for providing electric braking, the series motors have been altered by converting the series field-windings into shunt windings, a small series coil being added. The controllers have been changed by limiting the motion to the series notches. The controller acts to insert or cut out resistances in the armature circuit, as previously. In addition a "speed-control" lever is provided, by which resistance may be inserted into the field circuit. This handle is interlocked with the regular controller handle, so that the latter must be in the full-current position before the former can be moved, but this control-lever may then be advanced to insert successive resistances in the field, and thereby increase the speed. As soon as the control lever is returned to full excitation the main handle flies back to zero-current, unless it is held. When the line-current fails, the armature generates current which goes in reverse direction through the field and produces a braking action; an automatic switch cuts resistance out of the shunt windings when the line-current fails, to give maximum braking power, or the motorman may throw in the auxiliary series winding for the same purpose. When going down hill the motor is driven by the car and returns current to the trolley-line. This system, which has been patented by Mr. J. S. Raworth, is said to save considerable electrical energy over the ordinary series system.

A WIRELESS AUTOMATIC FIRE-ALARM TELEgraph has been devised by Mr. E. Guarini, of Brussels, Belgium. A thermostat or a mercury thermometer provided with appropriate contacts is arranged to close a battery circuit through a magnet, whenever the temperature around the thermometer rises above a certain point. The magnet, when energized, attracts an armature which releases a wheel driven by clock work. A brush presses against the surface of the wheel, and closes a battery circuit through the primary of a transformer whose secondary is shunted by a spark-gap. One end of the secondary winding is connected with an air-wire, the other end is earthed. This apparatus constitutes a transmitter. The rotating wheel has notches on its surface which interrupt the circuit so as to send out waves from the air-wire, and by properly spacing the notches a definite signal may be sent, as for instance a number representing the building in which the fire-alarm transmitter is located. At the fire-station is a simple coherer-receiver, consisting, for instance, of an air-wire connected to ground through a coherer which latter is shunted by a telegraphic relay. Then whenever the temperature in any of the buildings containing transmitters rises sufficiently to indicate fire, the fire-station receives a signal, whose characteristic, as above indicated, denotes the location of the fire. From the inventor's description of the device it appears that the transmitter will continue to send this signal until the temperature around the apparatus again drops below the critical point, i. e., until the fire is out. When this occurs the magnet releases the armature and the latter engages a detent on the rotating wheel, thereby stopping the transmitter.

THE BATTLESHIP "KING EDWARD VII.," the largest in the world, by a small margin, was launched at Devonport, England, on July 23. This ship is 16,350 tons displacement, or only 350 tons more than the U. S. battleship "Louisiana," now building at Newport News, and the "Connecticut," building at New York. The "King Edward VII." is 425 ft. long, 78 ft. beam and 26¾ ft. draft. Her engines will develop 18,000 I.HP. The Krupp steel armor belt ranges from 8 to 12½ ins. thick. The armament will consist of four 12-in guns, four 9.2-in. guns, ten 6-in. guns and twenty guns of smaller caliber. Her estimated speed is 12.5 knots, and she will carry 950 tons of coal and a crew of 800 men. The cost is given at $7,500,000.

THE CHICAGO RIVER IMPROVEMENT is discussed in the annual report of Col. O. H. Ernst, U. S. Engineers. During the past year no work was done by the government, but the Chicago Sanitary District continued the work. It dredged 476,344 cu. yds., built 5,078 ft. new dock wall, and completed five bridges. Congress has provided for two turning basins, and land is being acquired for these; one will be at the junction of the south and west forks of the South Branch, the other near the head of Goose Island, on the North Branch. It is expected that construction work on these basins will be commenced next year. The report calls attention to the decline of traffic on the Chicago River, and states that this is due to the obstructions, particularly those formed by the street railway tunnels. The decline will probably continue unless these obstructions are removed. In the Chicago Harbor the government continued its dredging, and the anchorage area (with 21 ft. of water) has been extended until it is now 79 acres in extent. Owing to the condition of the north pier or breakwater, the danger from fire and the high cost of timber, it is recommended that a new concrete superstructure should be substituted for the old timber work, at a cost of about $90,000.

THE ILLINOIS & MISSISSIPPI CANAL, or the Hennepin canal, cannot be finished for at least three years, according to the report of Major Willard, U. S. Engineers, of Chicago, in whose territory the canal lies. The delays due to litigation as to the bridges, and due to floods and stormy weather, are given as the reason for this dismal outlook. The canal will be about 75 miles long. It has been under construction since February, 1895, and seems likely to remain under construction for several years to come. Even when completed it will be already obsolete, as shown by a note in our issue of Sept. 4, 1902. Major Willard calls attention to special reports made by himself and by his predecessor, Major Marshall, on the subject of "making the canal of practical use for canal boats and barges of large capacity, whether with steam or animal power."

THE MOST SERIOUS RAILWAY ACCIDENT of the week occurred on the Chicago Great Western Ry., near Dodge Center, Minn., on July 26. A south-bound freight train misconstrued an imperfect train order and in consequence failed to take a siding to leave the track clear for a north-bound passenger train. A butting collision resulted, in which four persons were killed and many others injured.—On July 28 a work train out of control ran out from a siding onto the main track of the Southern Ry., near Springfield, Va. A south-bound limited passenger train collided with the work train, and in the wreck two persons were killed.

THE SPECIAL REPORT ON THE JEROME PARK reservoir, judging from the meager abstracts available up to the time of going to press, does not seem to sustain the serious charges which have been made against the character of the work thus far accomplished, nor the difficulties involved in securing a tight bottom. It has been alleged that certain retaining walls are very bad examples of masonry construction, and that the concrete bottom, as designed, would not hold water. The various questions involved were referred by Mayor Low to Prof. Wm. H. Burr, Rudolph Hering and John R. Freeman, who submitted a report on Tuesday, July 23. The report is now in the hands of the Aqueduct Commission. Further information on the subject will be given in a later issue.

A SERIOUS ELECTRIC RAILWAY ACCIDENT occurred on the Boston & Worcester Street Ry., near Westboro, Mass., on July 25. A well-filled car neglected its orders and passed a siding at which it was to meet an opposing car, so that a butting collision resulted. One person was killed outright and others may die.

A BAD RAILWAY WRECK IN ENGLAND caused the death of over a dozen persons on July 27. An excursion train entering the station at Glasgow ran into the bumping post with force enough to telescope two of the cars.

THE LARGEST AND HEAVIEST LOCOMOTIVES ever built in England for passenger service are now being erected at the shops of the Northeastern Ry., to the designs of Mr. Wilson Worsdell, Locomotive Superintendent. They are of the Atlantic type, with outside cylinders, and 6-wheel tenders. The leading dimensions are as follows:

Cylinders (outside)	20 × 28 ins.
Driving wheels (4)	6 ft. 10 ins.
Truck " (4)	3 " 7½ "
Trailing " (2)	4 " 6 "
Tender " (6)	3 " 9½ "
Wheelbase, driving	7 " 7 "
" " engine	28 " 0 "
" " tender	12 " 8 "
" " engine and tender	52 " 3½ "
Length over buffers	62 " 3½ "
Weight on driving wheels	87,300 lbs.
" " truck	36,000 "
" " trailing wheels	43,680 "
" of engine	168,080 "
" " tender	97,440 "
Boiler, diameter	5 ft. 6 ins.; length 15 ft. 10½ ins.
Boiler pressure	200 lbs.
Firebox, length	9 ft.
Heating surface, tubes	2,276 sq. ft.
" " firebox	180 "
" " total	2,456 "
Grate area	27 "
Coal capacity	5 tons.
Water in tank	4,950 U. S. galls.
Height, rail to center line of boiler	8 ft. 11 ins.
Height, rail to top of smokestack	13 " 1 in.

SOME TENSION TESTS OF PUNCHED PLATES are reported by Prof. E. J. McCaustland in an article in the "Journal of the Association of Civil Engineers of Cornell University." A number of steel plates, 6 × ⅝ × 30 ins. in size, had a series of ⅝-in. holes punched in them; the plates were then tested in a tensile-test machine, the extensometer being set on a 14-in. length. One of the plates had the holes filled with rivets before testing, and one plate without holes was tested. The ultimate strength figured on the net section was slightly increased, while the elastic limit, figured on the net section, was about the same as in an unpunched plate. The modulus of elasticity, figured on the gross section, is lowered. Some conclusions are drawn as to the strength values of a riveted member, but the data seem entirely insufficient for any conclusions in this regard. In the course of one of the tests, the holes in one row were drifted from ⅝-in. to ¾-in. diameter; this raised the elastic limit, as might be expected, but it also caused the final failure to take place in the adjoining row of undrifted holes. The same thing occurred with three different specimens.

WIDE TIRED WHEELS are provided for by an ordinance passed by the city council of Chicago on July 29, but the ordinance does not go into effect until Jan. 1, 1905, thus giving wagon owners 2½ years in which to make the necessary changes to their wagons. The width of tires required for wagons of varying capacity are specified in the ordinance, as follows:

Wt. Carried, lbs.	Width of tire, ins.	Wt. Carried, lbs.	Width of Tire, ins.
2,000	1	10,000	5
3,000	1½	11,000	5½
4,000	2	12,000	6
5,000	2½	14,000	6½
6,000	3	16,000	7
7,000	3½	18,000	7½
8,000	4	20,000	8
9,000	4½		

SIXTEEN-STORY CONCRETE-STEEL OFFICE BUILDING AT CINCINNATI, O.

The use of concrete-steel construction for buildings has made great advances within the past few years, and by far the boldest and most ambitious example of such construction is a sixteen-story office building now being erected at Cincinnati, O. This is the Ingalls Building, at the corner of Vine and Fourth Sts., and is owned by the Ingalls Realty Co. Railway ticket and telegraph offices, stores, and the entrance hall, will occupy the first floor; on the second floor will be a bank, while the whole of the top floor will be occupied by the Western Union Telegraph Co.'s operating room. character of the design being new to the officials of the building department of the city, some delay was experienced in securing approval of the plans.

Work was commenced in October, 1902, and it is expected that it will be completed by Dec. 31, 1903. By May 1, the seventh floor was completed and work was progressing at the rate of one complete story (concrete work) every 12 days. During the week of May 6, the eighth floor was under construction, and up to that date nearly 2,500 cu. yds. had been put in.

GENERAL DESCRIPTION.

The building is rectangular in plan, and occupies a site approximately 100 × 50½ ft. It is built upon a good stratum of gravel and sand, with foundations of the spread type, having large footings for the columns and walls, just below the basement level. The boldness of the structural design may be realized from the fact that the columns are so spaced as to require girders of 16 to 33 ft. span, and floor panels 16 × 33 ft. between the main girders. The height from floor to floor is 12 ft. 6 ins. in the office floors, and 17 ft. in the telegraph exchange on the 16th floor. The height from sidewalk to cornice is 210 ft. The use of concrete-steel construction enabled the height to be reduced by about 1 ft. for each floor, as compared with the height required for structural steel construction, this reduction being due to the shallower floors. Fig. 2 is a vertical section showing the girders and floors of the top and bottom stories.

The exterior face work is of 4½-in. marble to a height of three stories, above which it is of glazed light-grey or "granite" brick with terra cotta trimmings. The brick facing is supported at every floor by a ledge formed in the concrete, and is also secured by wire anchors projecting from the concrete, the wires being passed through holes in the forms before the concrete is deposited. The marble facing and the terra-cotta work are formed with grooves in the back, the top and bottom surfaces of which have an upward inclination. These engage with ribs of similar section on the face of the concrete, and form a species of dovetail joint, as shown in Fig. 3. The window and door frames in the exterior are of cast iron in the first story, and of sheet iron above. The sashes are also of metal. In the main panels of the second-story windows, 7 ft. high, the wide sashes turn on vertical pivots, while the narrower sashes are hinged at the side. In the upper panels or transoms, 36 ins. high, the sashes turn on horizontal pivots.

FIG. 1. THE INGALLS BUILDING, CINCINNATI, O. (CONCRETE-STEEL CONSTRUCTION).

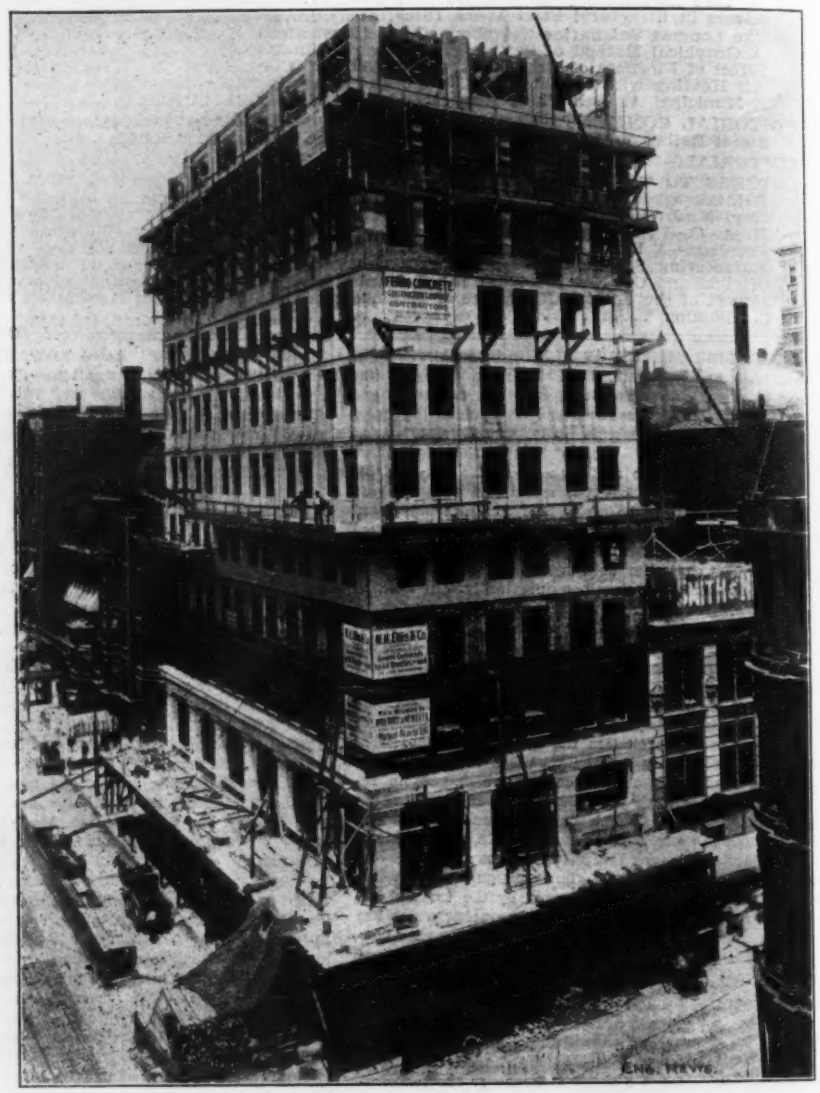


FIG. 15. VIEW OF CONCRETE-STEEL BUILDING, SHOWING WORK IN PROGRESS.

Elzner & Anderson, Architects and Engineers; W. H. Ellis & Co., General Contractors; Ferro-Construction Co., Contractors for Concrete-Steel Work.

Fig. 1 is a general view of the building, reproduced from the architects' drawing.

The architects are Elzner & Anderson, of 18 E. Fourth St., Cincinnati, and they conceived the idea of erecting a monolithic concrete-steel structure. They adopted the Ransome system, in which the concrete is reinforced by rods, bars, stirrups and hoops of twisted steel. Plain bars of circular section are also used in the columns. The general design and the detail plans, both architectural and structural, were prepared by Elzner & Anderson. The details of the concrete-steel construction were worked out by Mr. H. N. Hooper, the engineer of the Ferro-Concrete Construction Co., of Cincinnati, operating under the Ransome patents. The general contractors are W. H. Ellis & Co., of Cincinnati, and the concrete work is being done by the Ferro-Concrete Construction Co., Mitchell Building, Cincinnati. Mr. Albert Witte is Superintendent of Construction for the architects. The

There are four hydraulic passenger elevators, one of which is made specially heavy to carry freight, safes, etc. There are also two elevators from the sidewalk to the basement. One of these serves the boiler and engine room. The other is for handling freight, and from it a track extends across the basement to the freight elevator to the rear.

Besides the 16 office floors there are a sub-basement, a basement and an attic, the latter containing the overhead elevator machinery, and the water tanks for domestic supply and for the elevators. The sub-basement contains the boilers, two engine and generator units for electric light and power, elevator machinery, filter plant, etc. The space beneath the sidewalks is utilized, concrete-steel retaining walls being built under the curb lines of

the streets. The wall on the south side and part of west side is 14 ft. deep, but that on the rest of the west side is 21 ft. deep, there being a mezzanine floor between the basement and first floor.

These retaining walls are quite thin, and are reinforced by vertical beams or counterforts. They are braced near the bottom by the basement floor and at the top by concrete-steel girders or struts running back to the main columns and forming also the sidewalk girders. When the walls were built they were braced by shoring until the columns and sidewalk girders were built. The sidewalk is 4⅝ ins. thick, in panels 16 × 8 ft. between the girders, both girders and sidewalk being built at the same time and forming a monolithic mass. The girders are reinforced by steel bars, and the sidewalk has two layers of steel bars in contact and at right angles to one another. A light 1-in. metal lath ceiling is built at the level of the under side of the girders to prevent the dropping of water of condensation from the under side of the walk. This was built after the girders

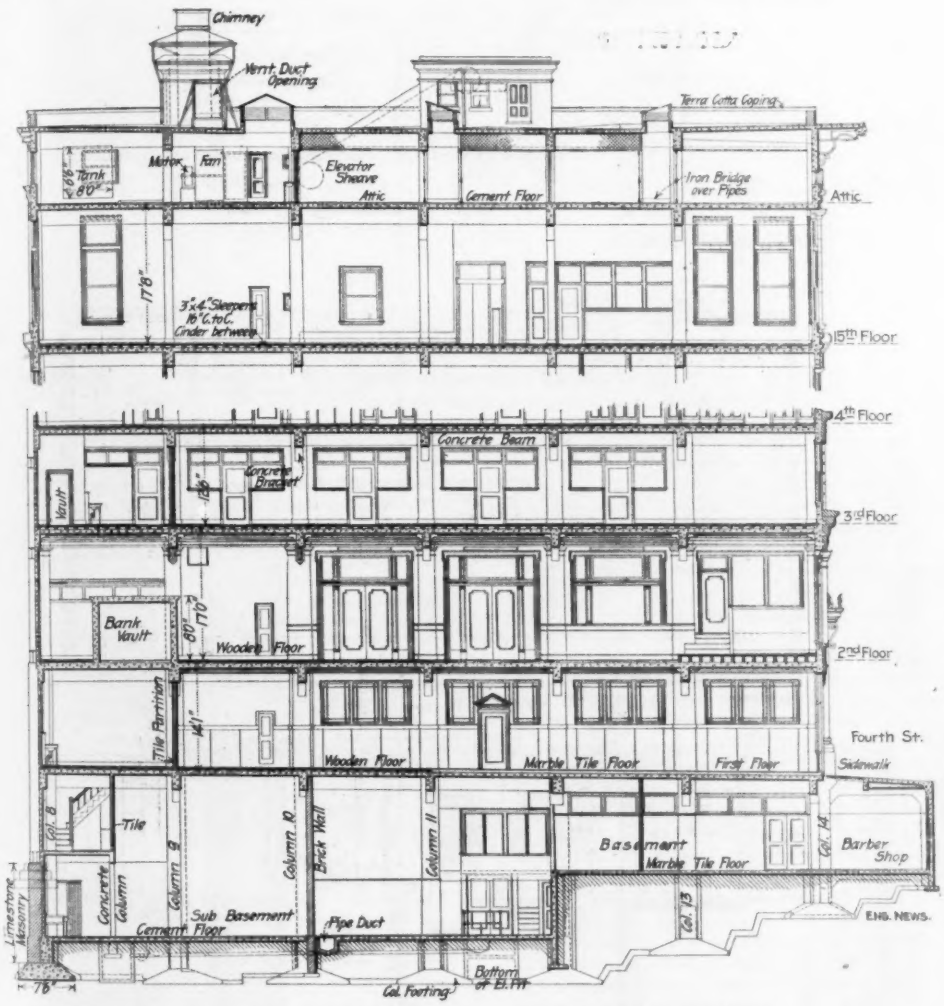


FIG. 2. PART VERTICAL SECTION; INGALLS BUILDING.

and walk, and its panels are supported by wire hangers embedded in the sidewalk concrete as it was laid and passing through holes in the bottom of the form. The retaining walls are shown in Fig. 4, and it will be seen that the 21-ft. wall has a narrow toe or footing on the outside and a wide toe on the inside, the latter acting as a brace or strut to resist overturning.

COLUMNS.

The columns are 16 to 33 ft. apart c. to c., as shown in the foundation plan, Fig. 5, and the main columns reduce in size from 34 × 38 ins. at the bottom to 12 × 12 ins. at the top. The footings vary in size according to position and loading, and Fig. 6 shows some of these footings, with the arrangement of the steel bars. The footings are built independently of the columns, and each has a rectangular pedestal somewhat larger than the column. Upon this is set a cast-iron base plate, having circular projections on top to form seats for the vertical round steel bars which act in compression only; the bases are left open on top, the spaces being filled by the concrete. The cast-iron bases for two of the columns are shown in Fig. 7. The tops of the projections are faced to a true horizontal surface so as to form a true bearing for the faced ends of the bars.

Each column has four, six or eight of these plain bars, 2 to 3½ ins. diameter, the joints of which are made by faced ends in contact. In the lower part of the building these bars extend through one story only, but above the third floor they extend the height of two stories. At each stage of the work a sleeve is put around the top of the bar to form a socket for the next bar. When ready for the next stage, the upper bars are set in position on the lower ones, and cement grout is filled into the sleeves as the concreting is carried on above the joint.

The circular bars are to carry compression loads, and in addition to them each column has four to ten smaller bars of twisted steel to take the tension due to wind load. The series of upright rods in each column is surrounded by rectangular hoops of twisted steel at vertical intervals of about 12 ins., the ends of each hoop being lapped and bound with wire, while the hoops are secured by wire ties to each of the upright bars with which they come in contact. The function of these steel hoops is primarily to resist the tendency of the compression rods to buckle outward. At the same time they greatly increase the shearing strength of the columns. The column loads, on the shoes, range from 500 to 750 tons. Fig. 7-A shows the arrangement of steel work in a typical column.

GIRDERS AND FLOORS.

The main girders between the columns are of 16 to 33 ft. span. Those in the first floor are 36 ins. deep; 34 ins. in the second floor, and 27 ins. deep above this. The width is 20 ins. The depth includes the thickness of floor slabs, which is 7 ins. on the first floor and 5 ins. above. They are made monolithic with the walls, columns and floors, and the ends of their horizontal steel rods project into the columns, fitting between the vertical bars of the latter. The girders are figured for fixed ends. Besides the steel rods, the girders are further reinforced by U-bars of twisted steel, set at varying distances apart; every other U-bar being set inverted. The girders are left exposed, except in the corridors, where a thin ceiling is built on the under side. The space between this ceiling and the floor is utilized as a ventilating duct, the ceiling being kept 6 ins. below the girders to give communication between the spaces of adjacent panels.

At the junctions of the girders with the columns, the work is strengthened by vertical diagonal bars extending from the top of the girder downward and from the bottom upward into the body of the column, the lower diagonals being concealed within concrete brackets and acting as wind bracing. The typical girder construction is shown in Fig. 8, which represents certain of the main girders on the fourth and fifth floors. The girders are all made with beveled top edges, running into the floors, avoiding sharp corners, and graduating from the heavy web to the thin floor flanges.

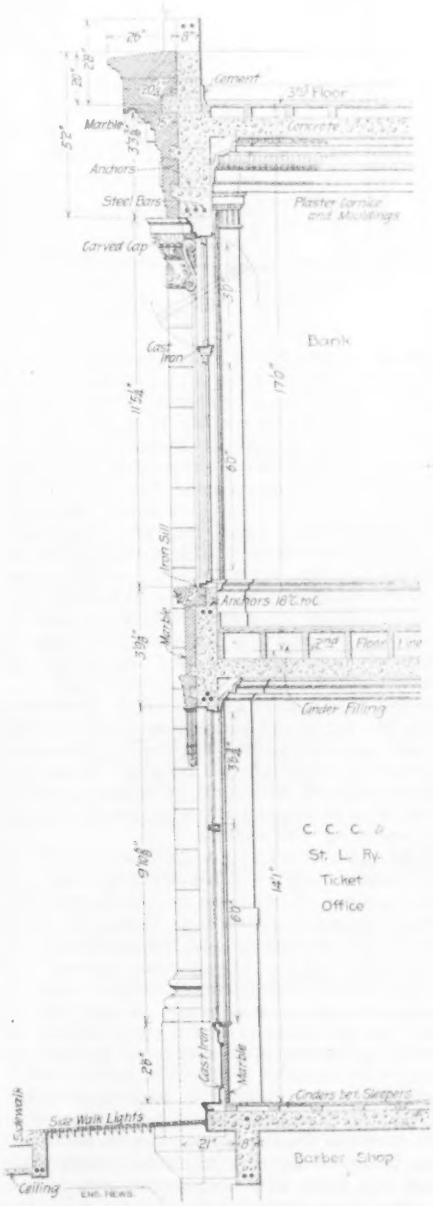


Fig. 3. Detail of Wall Construction, Showing Attachment of Facing to Concrete.

The following is the list of steel bars and stirrups or U-bars in the girders shown in Fig. 8:

Mark.	Bars Number.	Size, ins.	Length, ft.	Mark.	Bars Number.	Size, ins.	Length, ft.
1.	4	1¼	10	11.	2	¾	6
2.	2	1	13	12.	2	1¼	34
3.	2	1	9¾	13.	2	1¼	14
4.	2	1	34	14.	2	1½	11½
5.	2	1	34	15.	2	¾	5
6	2	½	7	16.	2	¾	8
7.	2	1	8¼	17.	2	¾	7
8.	2	1¼	10	18.	2	1	13
9.	2	1¼	11½	19.	2	1	16
10.	2	¾	5½				

U-bars in Left-hand Span:
4 Type X, ¾-in. 10 Type X, ½-in.....5 ft. 4 ins. long.
14 Type Y, ¾-in. .....7 ft. 8 ins. long.
U-bars in Right-hand Span:
10 Bars, ½-in.; length, 4 ft. 4½ ins.

The floors are 7 ins. thick on the first floor and 5 ins. elsewhere, the concrete being reinforced by a network of two layers of twisted bars laid at right angles. The bars are in contact, but not tied together. The main floor panels, 16 × 32 ft., are subdivided into two panels, 16 × 16 ft., by an intermediate cross beam, which is monolithic with the floor slab. These panels are figured as a flat floor slab supported on four edges. The lower floors and the corridors are covered with tile, laid in cement directly upon the concrete. The upper floors are covered with maple flooring nailed to wooden sleepers with cinder concrete filling between them. The floors are calculated for live loads of 60 lbs. per sq. ft.; except the first floor, 200 lbs.; and the second floor, 80 lbs. Fig. 9 shows two cross-sections, in opposite directions, and this shows the floors and girders.

The roof is of similar construction to the floors, and has the top of the concrete covered first with three-ply saturated roofing felt cemented together with coal tar. Over this is a layer of concrete 1½ ins. thick, finished with ½-in. of cement troweled smooth; this is blocked off with grooved joints which are filled with asphalt.

WALLS, PARTITIONS AND STAIRWAYS.

The exterior walls are 8 ins. thick (exclusive of the facing), but the walls coming against adjoining buildings are only 3 to 4 ins. thick. They are all reinforced by vertical and horizontal bars of twisted steel. Some of the partition walls are of mackolite, which is easily cut, allowing of removal or rearrangement in the future, as may be required. The partitions around the coal bins are of concrete, as they have to resist the pressure of the coal.

The typical arrangement of steel bars in the outer walls is shown in Fig. 10, which represents a portion of the Vine St. face. The vertical bars marked A are ½-in. bars 27 ft. long, extending through two floors, and having their ends lapped at the floor line. They are placed 2 ins. from the edges of the window openings. The vertical bars marked B are the column wind bars; joints are made at the middle of the window openings in each story, the ends being butted together. Over each joint is lapped a short piece of ½-in. bar or four ¼-in. bars of twisted steel. The horizontal bars marked C are pairs of ½-in. bars placed side by side near the outside and inside faces of the wall and 1½ ins. above or below the edges of the window openings. These bars are in lengths of 27 ft., and the ends of adjacent bars are lapped at least 21 ins. Where the joints occur over the window openings, between the columns, the lapped ends are wrapped with wire or secured together with hog rings.

One of the stairways is shown in Fig. 11. The concrete is 4 ins. thick at the back of the steps, and is reinforced by five lines of ½ in. bars and two ¼-in. bars, the latter being at the outer edge. The stairs have marble treads, risers and wainscot; and iron railings and newel posts.

PROPORTIONING OF CONCRETE AND STEEL.

The size of a column being determined (by architectural considerations and the loads to be carried), the bearing capacity of this section was first determined for concrete alone, and the deficiency provided for by round smooth bars of the required sectional area. Not only the actual loads to be carried, but also the strains due to shrinkage of the concrete had to be considered in the calculations. The stresses upon the concrete and the steel were made proportional to their respective moduli of elasticity.

The shrinkage or vertical contraction is less in a concrete-steel column than in a concrete column without steel. The ultimate adhesion of the concrete is taken at about 500 lbs. per sq. in. of surface contact with the steel, and single ½-in. twisted bars in concrete blocks 8 × 8 × 12 ins. pulled out from the concrete at 19,000 lbs. The twisting of the rods increases the adhesion of the concrete nearly 60%, and increases the tensile strength of the rods about 33%, as compared with untwisted bars; it also increases their elastic limit to about the ultimate strength of the untwisted bars.

The bars are twisted on the site by the contractors (the Ferro-Concrete Construction Co.), using a special machine designed and built by them. This machine has three different heads, which rotate at different speeds. For the heaviest bars the head having the slowest speed is used, and for the lighter bars, of course, greater speed can be used. The machine is run by an electric motor. In this machine only one bar is twisted at a time, but it can twist three at once if it was necessary. The time actually taken for the twisting is very short, and would not average more than one minute per bar. The bars are twisted so that the angle of the twisted edge is tangent to an angle of about 20°. In a 1-in. bar this is produced by

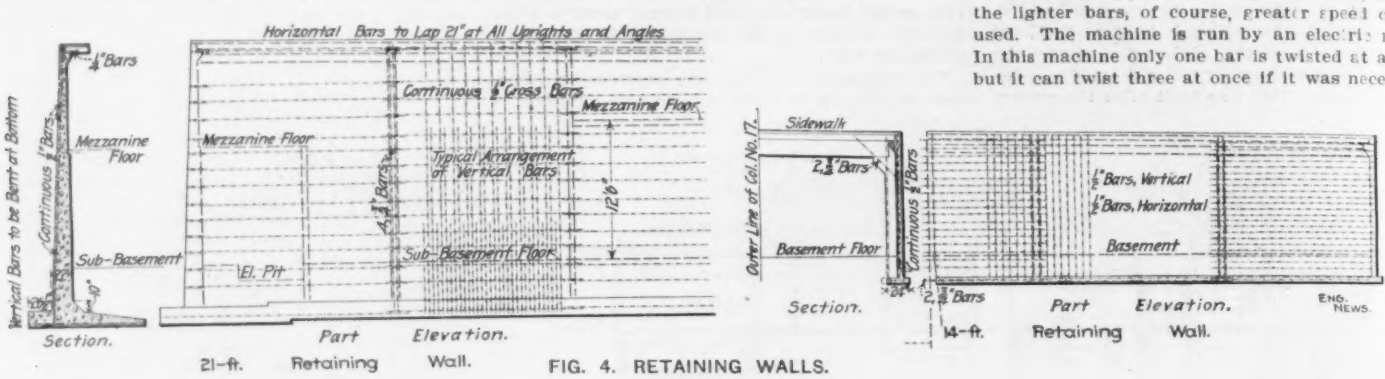


FIG. 4. RETAINING WALLS.

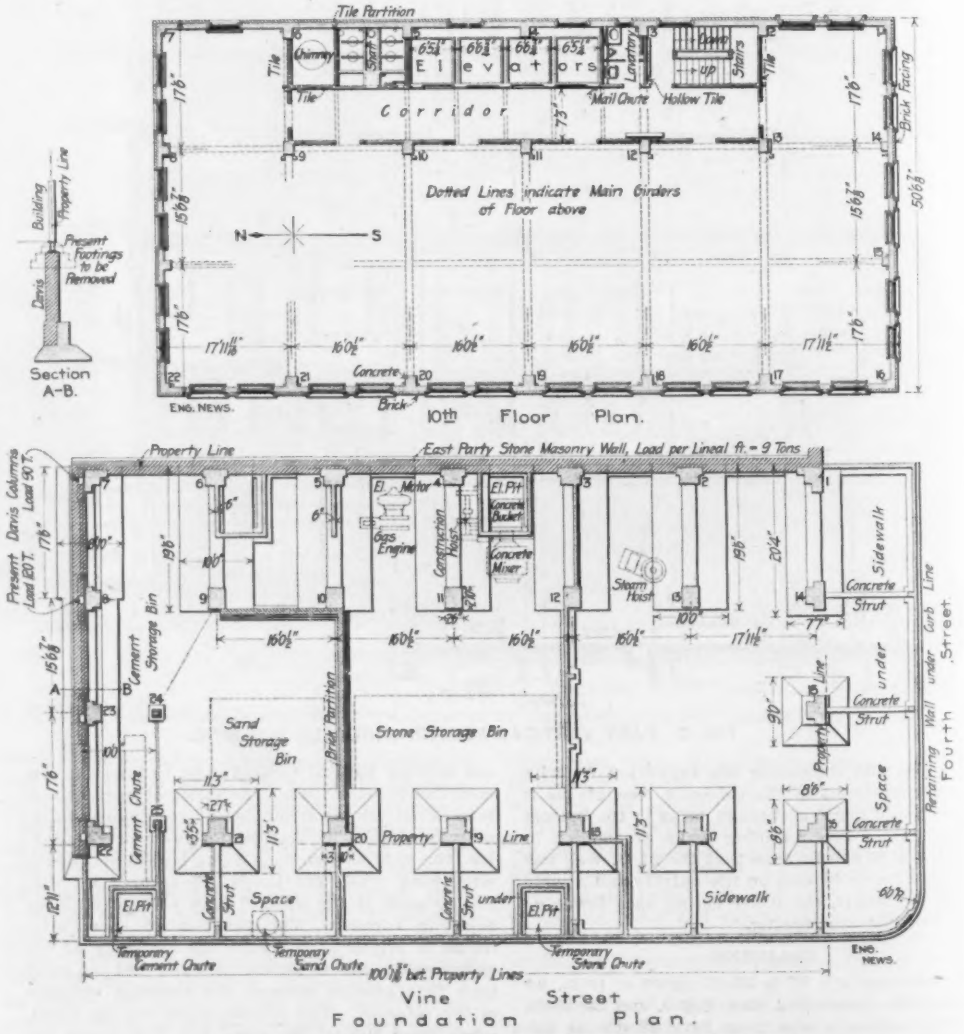


FIG. 5. FOUNDATION PLAN AND TENTH-FLOOR PLAN.

one complete turn in each 12 ins. of length. They are put in place and concreted with the ordinary light rust remaining upon them, but care is taken not to have any grease upon them when the concrete is deposited.

CONCRETE WORK.

The concrete is composed of 1 part of Portland cement, 2 parts of sharp sand, and 4 parts of 1-in. hard limestone or washed gravel. Below the street level the "Diamond" brand of cement was used; above this, "Lehigh" cement is used. The stone and gravel are both crushed, and the crusher-run product (without screening) is used. In measuring, two sacks of cement are used to one wheelbarrow of sand and two wheelbarrows of stone. These wheelbarrows are of iron, specially made for the work. They have two wheels to prevent tipping over, and to allow the contents to be readily discharged in front. The sides are level, so that the contents can be "struck" or leveled off, the struck quantities being used in measuring.

The concrete is mixed very wet, being just wet enough that it cannot be piled up in the wheelbarrow. The quantity of water is varied to suit the various purposes for which the concrete is to be used, and in close quarters where tamping is difficult, the concrete may be wet enough to allow the water to rise to the surface while the concrete is being wheeled to place. It cannot be tamped, in the ordinary sense of the word, but is stirred or worked in the forms; long iron rods or poles of 2 × 2-in. lumber are used; for the columns a long flat-ended paddle is used. This stirring of the concrete is done to allow any entrained air to escape, and also to ensure that the concrete thoroughly settles against the bars, filling all interstices around and between the bars. In the deep narrow girders, the concrete must be deposited slowly, and in small quantities, and be stirred at the same time, or it will be liable to arch or pack between the bars and leave voids.

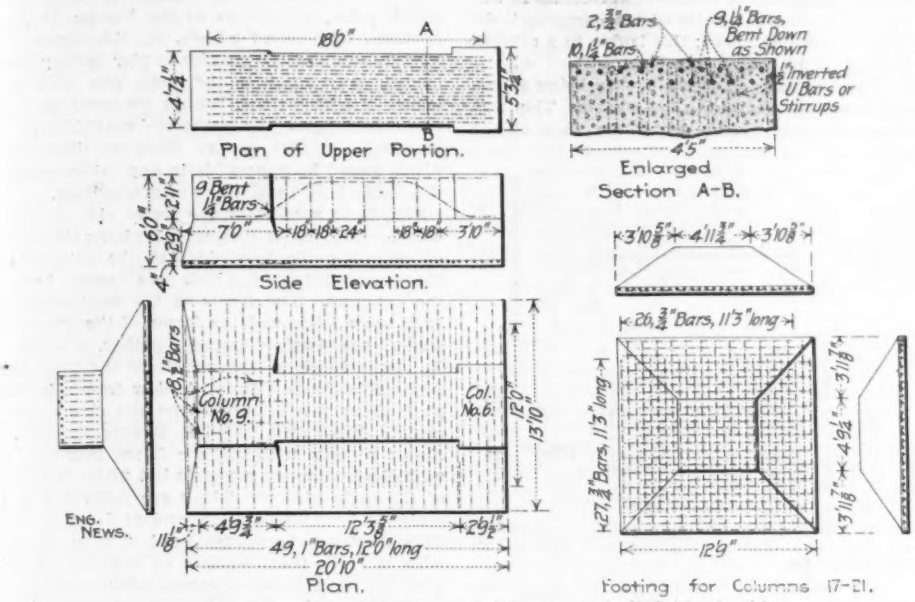


FIG. 6. FOOTINGS FOR COLUMNS.

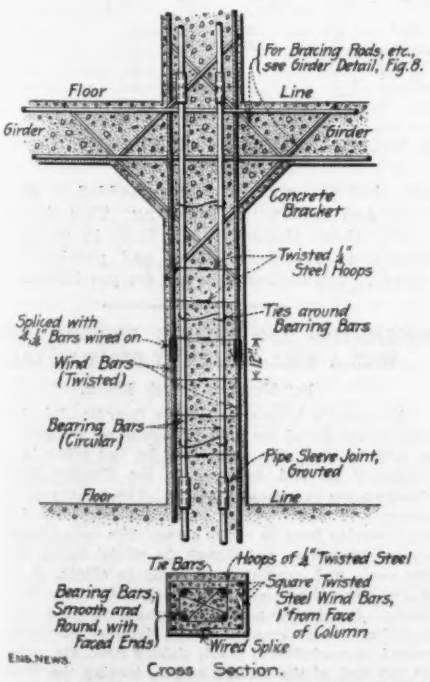


Fig. 7a. Typical Column Construction.

From plans furnished us by the Ferro-Concrete Construction Co., we have prepared the drawing Fig. 12, showing the arrangement of the construction plant, and this is also shown on the foundation plan in Fig. 5. In the sub-basement are a gas engine, an electric motor, a belted hoist and a Ransome rotary concrete mixing machine (Eng. News, March 8, 1900). These are so arranged that either the motor or the engine can be bolted to either the hoist or the mixer at will, by simply shifting the belt. Ordinarily the mixer is run by the motor, and the hoisting is done with a 20-HP. Lidgerwood engine. This engine also hoists all the lumber which is hoisted on the inside of the building. It is further used for hoisting all the steel rods, the majority of which are 34 ft. long, so as to extend over two spans; these are hoisted on the outside of the building by a boom derrick, which takes the rods directly off the wagons in the street. By having these three sources of power and two rigs for hoisting, it is almost impossible for breakdowns to cause delay or interference in the continuity of the concrete work.

The end elevation in Fig. 12, shows the operation of the hoisting bucket. This bucket is pivoted at the bottom, and so balanced that it always tends to fall forward. It has a capacity of 1 cu. yd. At the bottom of the elevator shaft, the guide is arranged at such an angle that the lip of the bucket is thrown forward under the chute of the mixer. When the bucket is filled it is hoisted up, the lip bearing against the front guide. At the floor where the concrete is to be used, the front guide stops, so that as the bucket is raised it automatically falls forward, dumping the concrete into a chute. This is fitted with a gate which can be opened or closed either by the wheelbarrow man or by the signal man who controls the operation of the hoist. The self-dumping bucket and special wheelbarrow already mentioned were furnished by the Ransome Concrete Machine Co., of New York. In building the footings, the mixer was placed a little below the street level, and the concrete discharged into chutes leading direct to the footings, but this practice was discontinued.

The cement, stone and sand are stored in the sub-basement, being hauled in dump wagons and emptied through openings left in the sidewalk. The two first are dumped through the sidewalk elevator openings, while a special temporary opening is provided for the sand, as shown in the foundation plan. An inclined chute for the cement prevents injury to the sacks. The sand and stone bins can hold about 30 carloads of material, and enough is always kept on hand for two floors in advance. From the bins extend runways sloping down towards the concrete mixer, so that the loaded wheelbarrows of sand and stone can be handled with the least effort and the greatest expedition. In this way the material is delivered largely by gravity from the dump wagons to the mixer. The cement is dumped into the mixer by hand, and this machine mixes the concrete in batches of about 2 cu. yds.

In building the floors, the steel rods are first laid in place, being blocked up from the form by smaller rods to give ⅜-in. clearance under them. The spacing is regulated by a scribed rod or board placed against the form. Upon these bars are laid others, at right angles to the first, and concrete is then filled in over them to the required thickness. The girder forms or molds are filled with concrete at the same time, the steel bars having already been placed in position within them. For the various pipes for plumbing, etc., sleeves of sheet metal are set in place upon the forms for floors, and filled with sand to prevent the concrete from filling them as it is deposited. The sleeves are carefully located and the pipes afterwards slipped through them. Outlet boxes for electric wiring are also set in position and embedded in the concrete as the work progresses.

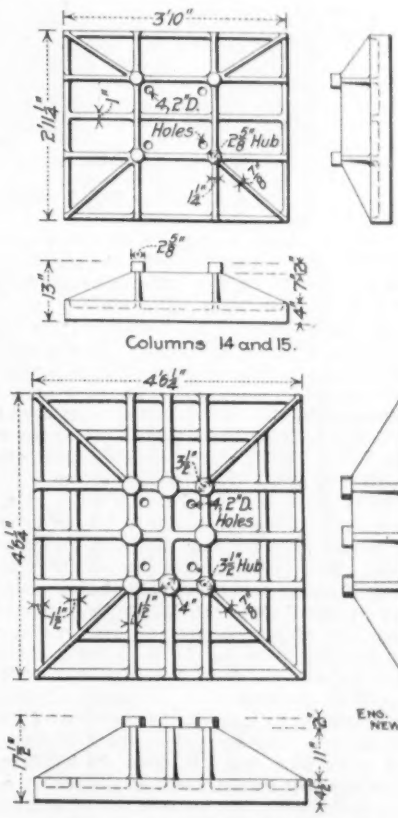


Fig. 7. Cast-Iron Bases for Columns.

Fig. 13 shows the work in progress, with floor bars laid and a wheelbarrow load of concrete just dumped. In the girder just filled at the right will be seen the stirrups, while at the left is the group of round and twisted bars for one of the columns. Fig. 14 shows the condition of the

work in February, with concrete finished up to one floor, and the forms and bracing in place, while the column and girder forms are being erected for another floor. This view is looking east from the Vine St. side. At the rear of the site is the concrete hoist, with the bucket dumping into a chute leading to where the work of concreting is in progress at the corner. In front is the derrick used for handling the steel bars and other material.

The forms are built on the site, the lumber being cut to size by an electrically-driven sawmill installed on the second floor below the one under construction, the mill being moved up from floor to floor as the work progresses. The column forms are heavily bolted and braced to resist the hydrostatic pressure, and the wall forms are supported by shores and braces for the same purpose. The party walls on two sides serve as the outer sides of the wall forms up to the height of their respective buildings. The forms and bracing are left in place for about a month. The column and wall forms are built in shifts of three stories, and are removed in heights of one story at a time, working from the bottom. The forms are struck on the outside with mallets to detect the existence of large voids and to ensure against the appearance of small voids on the surface.

The laborers are colored men as a rule, and the work requires constant supervision to ensure systematic progress and thorough filling of the forms. Work was done at temperatures as low as 28° F., some salt being used in the water, and the materials being heated. At low temperatures a number of salamanders or open stoves were kept burning under the forms in the basement to keep the concrete from freezing. No work was injured by freezing.

In setting out the work, the positions of the column centers were marked at the first floor level with tacks on a line 4 ft. from the building line. These were set by a transit, and tacks were then set near the top of the walls of the adjacent buildings, from which the position could again be laid off if the first tacks were disturbed.

It takes about ten days to put up all the forms for a floor, and two days to place all the concrete of the columns, girders and floor of an entire story (or 120 cu. yds.), so that the contractors are able to average 2½ stories per month. The rate could be increased, but there would be a probability that the tamping would be neglected.

Since the foregoing description was prepared we have received from the architects a set of photographs taken on May 9, and three of these are reproduced in Figs. 15, 16 and 17.

Fig. 15 is a general view of the exterior. This shows the platform over the sidewalk and the white marble facing in course of erection. It will be noticed that the exterior of the concrete looks dark up to the third floor; this is due to a coating of asphalt paint which has been applied with a view of preventing any possible stain being transmitted from the concrete to the marble. This was a simpler method than coating the back of the marble, but the architects think that it was an unnecessary precaution, since the concrete has thoroughly dried out and hardened long before the marble is applied. In this view will be noticed the concrete ledges at each floor line, for supporting the face work; also the various anchors built into the concrete and projecting ready for the face work. The marble extends as high as the third floor, and above this point are innumerable wire anchors built into the concrete, which are invisible upon the photograph; these are built in to anchor the brick facing. Further up in the building is a tarpaulin hung in the scaffold; this is to catch any drippings which are apt to occur when the concrete is being placed, and which being saturated with cement might stain the marble work below, besides greatly inconveniencing the workmen. This precaution works very well. It will also be noticed that balconies are carried all around the scaffolding at the floor level for the use of workmen, not only in erecting the forms, but also in striking the forms with mallets (as already described) so as to insure the concrete thoroughly settling in all parts. Above the building rises the scaffold of the concrete hoist, which is kept up two stories in advance of the work. The vertical wind rods are shown projecting above the walls.

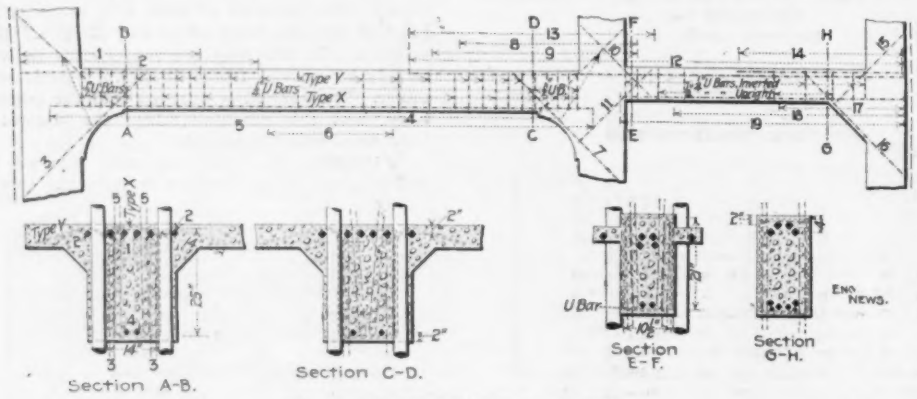


FIG. 8. CONCRETE-STEEL GIRDERS.

By referring to the platform over the sidewalk, it will be seen that the derricks used to hoist the marble into position lean against the building, at which point, as well as at the bottom, they are provided with small wheels. By this means they may be run along from pier to pier as may be required, very much in the same way as a shelf ladder in a book store. Within the openings of the two floors appear the plasterers' scaffolding. The plasterers in fact are at work on these floors which would be impossible in any other construction at such an early stage of proceedings.

Fig. 16 is a view of the under side of a floor which is just being concreted. It shows the method of clamping the four sides of the column, and the diagonal braces which are used between the columns. The forms of the floor above are built upon joists and the forms of the beams and girders are simply clamped together, as shown at A. The sketch in the corner of the view illustrates this simple clamp. A blow from a hammer at B from the right tightens the clamp, and a similar blow from the left loosens it. Fig. 17 shows a floor with all the forms removed, but with the temporary supports left under the center of the main girders. These are permitted to remain as long as possible, and at least 30 days. This may not be necessary, but it is certainly wise to leave them as long as possible, so as to give the concrete every opportunity to get its ultimate strength. The pipes shown are electric wire conduits connected up to the bends which were built into the floor. The floor in the view being the top surface of the rough concrete. The black spot seen through the support is an electric switch box built into the pier between the two windows, this switch operating the ceiling light attached to an outlet box embedded in the concrete and flush with the ceiling. This shows how nicely these things are all built in during construction, avoiding cutting and patching afterwards. The hollow tiles are for partitions.

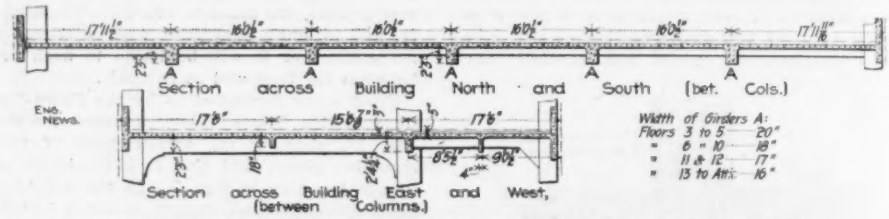

Section across Building North and South (bet. Cols.)
ENG. NEWS.
Section across Building (between Columns.) East and West.
Width of Girders A:
Floors 3 to 5 20"
" 6 " 10 18"
" 11 & 12 17"
" 13 to Attic 16"

FIG. 9. CROSS-SECTIONS OF INGALLS BUILDING, SHOWING FLOORS AND GIRDERS.

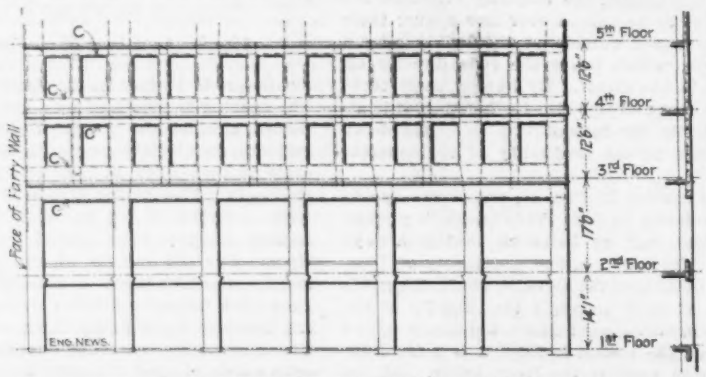


FIG. 10. STEEL-WORK IN OUTER WALLS.

PROSPECTING AND VALUING DREDGING GROUND WITH A WELL-DRILLER AT OROVILLE, CAL.*

By Newton Booth Knox.†

The gravelly bottom land below Oroville, Cal., is being extensively mined for gold by means of dredgers. Upon a strip of land nine miles long by two miles in width, bordering on and adjacent to the Feather River, 21 dredgers are at present working. These dredgers are the property of about twelve companies, who own dredging land varying from 80 to 800 acres. The companies are all close corporations, and though the official figures of working costs and profits are difficult to obtain, it is well known that they yield their stockholders excellent dividends. For instance, one company's working expenses, including labor, repairs, power, interest on capital invested, depreciation of plant, sinking fund, etc., amounted to but 32% of the bullion output, leaving the remaining 68% clear profit.

The total general average of the drillings of several of the largest companies' holdings, covering a period of several years, gives the value of the gravel between 17 and 19 cts. per cu. yd., and an average depth of 11 yds. As a dredge handles from 1,200 to 2,000 cu. yds. per day, at a cost of from 5 to 8 cts., average 6 cts. per cu. yd., it is evident that the returns in this work are considerable.

HISTORY.—In the early days of California history, Oro-

*Slightly condensed from a paper read June 18, 1903, before the Institution of Mining and Metallurgy.
†Assoc. M. Inst. Mining and Metallurgy.

ville district was extensively mined by first the white men and later by the Chinese. It is claimed that $82,000,000 has been taken out within a radius of eight miles of the town of Oroville. The gravels were worked to water level, and washed mostly in hand rockers. Pits and small shafts were sunk until a rich pay streak was encountered, which was followed, drifted out, hoisted to the surface and rocked. Water interfered greatly with this style of

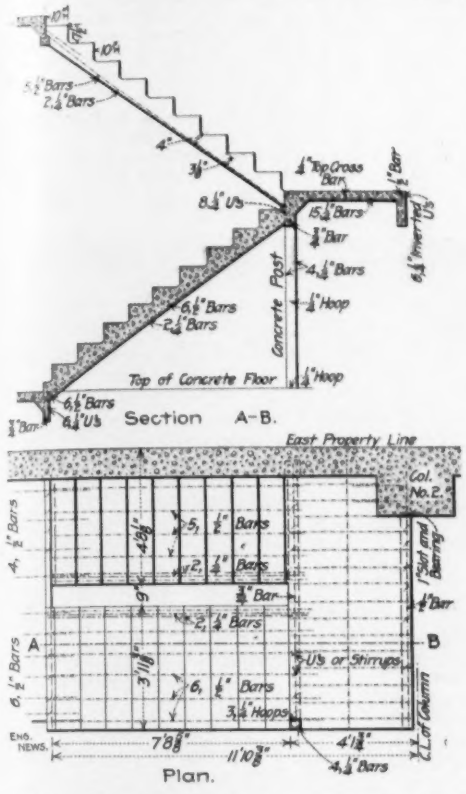


Fig. 11. Stair Construction in Ingalls Concrete-Steel Building.

mining, and only the richest streaks could be worked. The gravels left made excellent dredging ground.

THE GRAVELS.—The gravel deposit about Oroville evidently represents the flood plain of an ancient stream of the Sierra Nevada, near the point where it debouched into the gulf which then occupied the present Great Valley of California. As stated, the gravels average about 33 ft., varying from 28 ft. to 60 ft. in depth. A false bedrock of lava ash, or volcanic tufa, varying from the consistency of chalk to that of firm sandstone, is encountered at this depth. Below this false bedrock are gravels and sand. In one place a hole was drilled 80 ft. below, and showed sharp white sand. In some places the gravel is topped by a red soil overburden, varying from 10 ft. to 25 ft. in thickness. This overburden, and the material below the false bedrock, contain little or no gold. The gravel is fairly coarse, large boulders, weighing 300 lbs. or 400 lbs., being very exceptional. One-half of the gravel will pass through a ½-in. screen. As a rule the gravel is not cemented, though some dredgers have encountered pieces of ground which had to be loosened with powder.

The gold is distributed more or less equally throughout the gravels. Rich concentrated streaks of from an inch to a foot in thickness occur, followed by barren sandy areas, which seem to represent varying conditions of flow in the depositing stream. The gold is in a very finely-divided state, and easily caught by the riffles, cocoa-matting, and quicksilver of the dredge. Nuggets are a rare occurrence. The gold found in the bore holes is 922 fine, and worth $19 an oz. The dredge bullion is somewhat lower grade, the base being the surprisingly large amounts of lead in the form of quail, rabbit and duck shot, which the dredge digs up. On the Butte dredger they have collected about 50 lbs. of shot from about two acres of gravel.

The Oroville district is peculiarly favorable for dredging, for the following reasons: (1) Bedrock (so-called) is soft and easily cut by the dredge. (2) Boulders are small. (3) Water is available. (4) Power (electric) is cheap. (5) Gold is fairly evenly distributed. (6) Climate is such that work can be carried on 12 months of the year. (7) Labor is cheap and efficient; and finally (8) the gravels contain enough gold.

TESTING THE GROUND.—In order to ascertain whether a piece of ground is worthy of consideration, the following system is adopted: The land is first divided, according to its area, into blocks of from 5 to 10 acres, and a hole is drilled in the center of each of these blocks.

The Keystone driller has been generally adopted in Oroville as having proven itself fully equal to this work. The driller consists of a "walking beam" operated by steam power producing the required motion for raising and dropping the drill. In addition to this is a reel, on which is wound the rope of the sand pump used in pumping out the holes. The drill is suspended by a 2-in. in diam. Manila rope, which passes under one sheave on the walking beam over another to the main drum. As the walking beam moves, the rope is alternately tightened and loosened, which raises and drops the drill. The drill itself consists of: (1) Rope socket. (2) Drill stem—a piece of soft steel about 4 ins. diam., and from 12 to 15 ft. long. (3) The bit, with a single cutting face about 5¾ ins. long. When ground containing coarse boulders is being drilled, there is a liability of the drill becoming wedged in the hole, and in order to prevent this, a tool called the jars is inserted between the rope socket and the steam. It is like two links of a chain, and when the bit is caught these two links coming together with a shock jar the drill loose. The whole string of tools, socket, jars, stem and bit weigh about 1,200 lbs.

SPEED OF DRILLING.—The driller makes about 60 strokes (30 to 40-in.) per minute. About a half cord of oak wood is burned per day of ten-hour shift. For drilling and washing the drillings about 1,000 gallons of water is used per day.

THE OPERATION OF DRILLING.—The land is surveyed, and the location of the drill holes staked out and marked by flags. The drill is set up over flag No. 1. A hole is shoveled out to a depth of about 2 ft., and the first length of casing is made ready. This section of casing or drive pipe is 5⅛ ins. inside diameter, ⅜-in. thick, and about 5 ft. long. It is fitted with a wrought-steel drive shoe to protect the lower end from injury. The drive shoe is made 7⅝ ins. in diameter at the cutting edge, and is slightly beveled inward. On the other end of the casing is screwed a steel driving head to prevent the threads from being battered during driving. This casing so mounted, is set up in the hole directly under the suspended drill and tamped around with excavated dirt.

Drilling is now commenced. Water is poured in from time to time to thin out the material. After the ground below the foot of the shoe is loosened for a distance of, say, 1 ft., the casing is driven down and the loosened and thinned material removed by means of the sand pump, a hollow pipe supplied with plunger and foot valve.

The driving is accomplished by striking the driving head with a couple of iron blocks clamped to the stem by means of two 1¾-in. bolts, the weight of the string of tools acting as a hammer. After driving, the driving blocks are removed. When the first length of casing is driven down to head, the driving cap is removed, a second section of casing is screwed on the first, the driving cap replaced, and drilling resumed. When the required depth is reached, determined either by striking "bedrock" or passing through the pay stratum, the hole may be considered finished, and the next step is to pull up the casing.

This is accomplished by removing the bit, stem and jars, and replacing them by what is known as the pulling or pipe jars. These consist of an iron boss fixed to the end of a rod 4½ ft. long. Above the boss is a 1-in. thick plate—the "knocking head"—provided with threads which are screwed into the sleeve of the top section of casing. The stem of boss passes through a square hole in the plate. The walking beam is set into motion, and the string of casings is raised by the boss striking against the knocking head. As each section of casing is raised, it is unscrewed, and the knocking plate screwed on the next. If care is used in keeping the threads of the casing clean, the casings can be used for a long time. It is rarely that a casing is lost.

TREATMENT OF DRILLINGS.—The drillings extracted from the drill-hole by means of the sand pump are discharged into a wooden trough, 12 ft. × 1 ft. × 1 ft., set on a slight grade. From the trough they are run into the riddle of a rocker, and rocked in the ordinary method adopted for washing gravel. Great care is taken to save all the extremely fine particles of gold, as upon this work depends the accuracy of the tests made. It is customary to clean up results of each pumping, and to carefully note the number of countable colors obtained, and the character of the ground drilled through.

The term "fine gold" is applied to such specks as are too small to be counted, but which play an important part in making up the total value of the hole. The gold from each clean-up is put in a small dish.

This practice of cleaning up after each pumping (approximately after each foot of hole drilled) instead of one final clean-up is all-important in furnishing data for a cross-section map, showing occurrence of rich streaks—sandy or clay patches, fine gravel, depth of overburden, of false bedrock, and of water level.

After the last clean-up all the gold from the hole is collected by means of quicksilver forming an amalgam. This amalgam is dissolved in nitric acid and thoroughly washed in hot water. A few drops of alcohol added to the wash water will prevent the spattering and loss of gold when the last drop of water is evaporated. The gold is annealed and carefully weighed. From this weighing the value of the ground at this particular spot is calculated, and the result given in cents per cubic yard.

CALCULATING THE VALUES.—This is best shown by an example. The gold from, say, hole No. 14, weighed 2.22 gr.; this at 3.95 cts. per gr. equals 8.76 cts.—the value of gold from hole No. 14. The cubic contents of the whole is next calculated. To do this, a factor called the "pipe constant or pipe factor" is applied. The inside diameter of casing is 5⅛ ins.; the outside diameter is 6⅛ ins. It is the practice of the district to use the outside diameter of the pipe as a basis for calculating its contents, the local engineers holding that it is the displacement of the pipe and not the cubical contents that should be used. Figuring the cubic contents per ft. of pipe, with a diameter of 6⅛ ins., would give .23 cu. ft. In practice it is found that .23 is much too small, giving values too high, values not borne out by subsequent dredging.

Some engineers use .25 as a factor. "Radford's Factor" is .27—a factor obtained by Mr. W. H. Radford, a mining engineer of wide experience in this class of work by the following method: Mr. Radford sunk a shaft 3 ins. in diameter, using a drill hole as the center, to a depth of 34 ft. The gold obtained from the shaft corresponded almost exactly with the gold obtained from the drill hole when using .27 as the factor in the calculation. This factor is very important, as on it depends the value of the holes and consequently the final value of the ground. The difference in the results obtained by using either the factor .25 or .27 is sufficient to change the value per cu. yd. from net to gross, i. e., a difference in some cases of 6 to 8 cts. per cu. yd.

Continuing the calculation: Hole No. 14 was 29½ ft. when drilling was stopped; 29½ × .27 gives 7.965 cu. ft. in the hole drilled. Now we have the simple proportion, 7.965 cu. ft. of gravel drilled : 27 (the number of cu. ft. in a cu. yd.) :: 8.79 (the value of gold obtained) is to X = the value per cu. yd., whence, X = 29.69 cts. per cu. yd.

The value of each hole in cents per cu. yd. is multiplied by its depth in feet, and the sum of the products divided by the sum of the depth; the quotient is the average value in cents per cu. yd. Thus, we have a block of dredging ground drilled with one hole to every 10 acres.

The sum of the products obtained by multiplying each depth by its corresponding value = 14.046. The sum of the depths is 660 ft. Dividing the first by the second gives 21.28 cts. per cu. yd. = average value of this ground. The average depth is 33 ft., or 11 yds.

At an average of 11 yds. deep this ground will contain 53,240 cu. yds. to the acre, and allowing 3,240 cu. yds. for ridges and "corners" left in dredging, we have 50,000 cu. yds. per acre, or 10,000,000 cu. yds. in the 200-acre

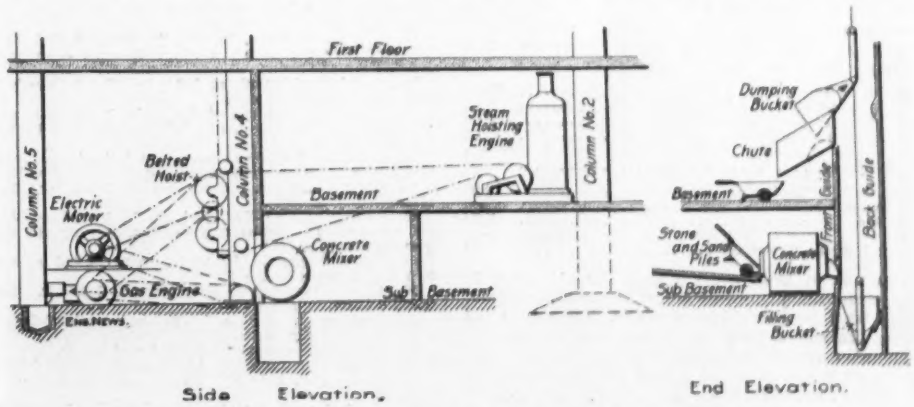


FIG. 12. CONSTRUCTION PLANT USED IN INGALLS BUILDING.

tract under consideration. This at 21.28 cts. per cu. yd. gives a total contents of $2,218,000.

Assuming the price of land is	$150,000
Cost of a 5-ft. bucket-ladder dredge	65,000
Working capital	10,000
Total cost	$225,000

A 5-ft. bucket dredge is rated at treating 65,000 to 70,000 cu. yds. per month, but up to the present time, the best these dredges have been able to do has been about 70% of the possible running time. It may be added that the new dredges now building hope to better this, and will doubtless bring up the running time to 80%. In fact a new dredge, the "Butte," which has been running but three months, has, during that time, averaged 75%, and the last month of their run they have averaged 86% of the time. Assuming 70% running time, a 5-ft. dredge will work out this ground at the rate of an acre per month, or in all, about 17 years for the tract. The working expenses of the dredge is as high as $3,500* a month, or $42,000 per year, and 1 ct. per cu. yd. may be assumed as lost in tailings.†

COST OF DRILLING.—The running expenses of a driller per day are:

Labor, 1 Driller	$3.50
1 Fireman	2.50
1 Teamster and team	3.50
Chinese rockerman	1.50
Total	$11.00
Wood, half cord, at $6	$3.00
Repairs, say	5.00
Drill hire	5.00
Total expenses per day	$24.00

Drilling about 10 ft. a day gives a cost of $2.40 a ft. Drilling contracts at Oroville are let for $2.50 per ft., and considering delays, breakdowns, repairs, road cutting and moving from hole to hole, this figure is reasonable enough. The number of feet drilled per day varies greatly—and depends on the character of ground and the season. In soft ground 20 to 30 ft. a day can be drilled with a corresponding reduction in cost per foot. In winter and spring, when the top soil is wet and soft, the problem of moving the machine from hole to hole, and of bringing in wood and water becomes a serious one. Just to show the extra difficulties, delays and stoppages a driller must contend with, I quote in full the log of Mr. Radford for the month of December, 1902:

1. Started hole No. 12, but did not do much owing to lack of good driving cap and scarcity of water. Made 14 ft.
2. Drilling on hole No. 12 all day. Made 7 ft.
3. Started about 7.30 a. m. Broke sand pump about 8.15. Had to take it to town to get it repaired. Finished hole No. 12 at noon. Commenced pulling casing 12.50. Finished and moved to hole No. 18—1,320 ft. Set up and started drilling.
4. Worked all day on hole No. 18. Made 20 ft.
5. Wet, did not work.
6. Started work on No. 18 after lunch. Made 7 ft.
7. Wet, did not work.
8. Worked on hole No. 18. Made 11 ft. Started pulling up casing.
9. Finished pulling up casing from hole No. 18, moved to hole No. 20 about 900 ft., ground very soft, had to raise drilling machine with jackscrews, and to keep it up by means of wood placed under the wheels. Made 14 ft. in hole No. 20.
10. Wet, did not work.
11. Wet, did not work.
12. Worked on hole No. 20.
13. Finished hole No. 20, 34 ft. deep; pulled up casing and started moving to hole No. 14—ground very soft.
14. Moved to hole No. 14, and set up machine, ground very bad in places; distance from No. 20 to No. 14, 1,320 ft.
15. Working on hole No. 14, made 17½ ft.
16. Working on hole No. 14; reached depth of 28 ft., broke bolt in driving blocks, got new one from machine shop in town. Lost 2 hours.
17. Finished hole No. 14, 29½ ft. deep. Pulled up casing and moved machine a short distance towards hole No. 16.
18. Moved machine about 100 yds. Got into soft mud and stayed there all day.
19. Raised machine out of the mud. Took off gear-ring on account of broken teeth, and took it into town and got it repaired.
20. Got gear-ring about noon, put it back on machine, and started, ran short distance towards hole No. 16.
21. Moving machine to hole No. 16, road very bad in places.
22. Moving to hole No. 16, 1,320 ft. from hole No. 14. Set up machine, and started drilling. Struck old drifted ground. Made 21 ft.
23. Finished hole No. 16, 20 ft. deep. Pulled up casing, and moved toward hole No. 10.
24. Finished moving to hole No. 10, 1,320 ft. Set up machine and drilled 4 ft.
25. Working on hole No. 10, reached depth of 19½ ft.
26. Wet, did not work.
27. Ground too soft to haul wood and water.
28. Working on hole No. 10; reached depth of 31 ft.
29. Working on hole No. 10; reached depth of 41 ft.—stopped.
30. Finished pulling casing from hole No. 10 soon after lunch; started moving to hole No. 8.
31. Finished moving to hole No. 8, 1,320 ft. Set up machine and started drilling. Broke walking-beam arm, had to send to town for a new one—lost 3 hours. Made 13 ft.

Although the crew worked a ten-hour shift during this month, only 7 holes were finished, giving a total depth of 212.5 ft., or an average of 6.8 ft. a day.

I quote this, not as an example of average work, but merely, as I have said, to show the difficulties—particularly in winter.

VALUE OF THESE TESTS.—Three years' drilling followed by subsequent dredging of the drilled tract has proven that these tests are but fairly indicative of the values in the tract. For instance, one company, in order to prove the efficiency of these tests, drilled an acre of ground with 23 drill holes, which afterwards dredged within 95% of the calculated value. On the other hand, I have heard that a company with a recently finished modern dredge has already dredged about 6 acres, which produced 35% of the drilled values (2 holes to the a[illegible]. It is unfortunate that more exact data regarding [illegible] values of these tests are not at hand. Engineers [illegible] recognize the ever-present liability of error and [illegible] sequent dissatisfaction arising from all work where [illegible] value of the large is calculated up from the value of [illegible] small. So the chief value of this method of test[illegible] ground lies not so much in proving the total pos[illegible] yield of a piece of ground, or its values in sight, but [illegible] indicating the presence and occurrence of the pay ch[illegible] nels, depth of bedrock, water level, etc., etc. It is [illegible] the custom to keep a driller well in advance of [illegible] dredge, the results of the drilled holes serving as a g[illegible] for its future movements.

I am greatly indebted to Mr. Wm. H. Radford, of S[illegible] Francisco, for much of the data in these notes. T[illegible] methods of cleaning up, and the calculation of valu[illegible] herein described are his. I am also indebted to th[illegible] Keystone Driller Company, of Beaver Falls, Pa., fo[illegible] their kindness in furnishing me with cuts of their tools and drilling machine parts.

*$2,500 per month is closer to a fair average of the month's expenses even when allowing a sum of $500 for possible repairs.

†A test has lately been made by Mr. W. H. Radford in the "Butte" tailings which were found at the end of the sluices, and which represent say 50% of the bank gravel. From 100 buckets of this material 0.6 ct. in gold and a little quicksilver, about the size of a peppercorn, were obtained.

FIG. 16. FORMS FOR COLUMNS AND GIRDERS.

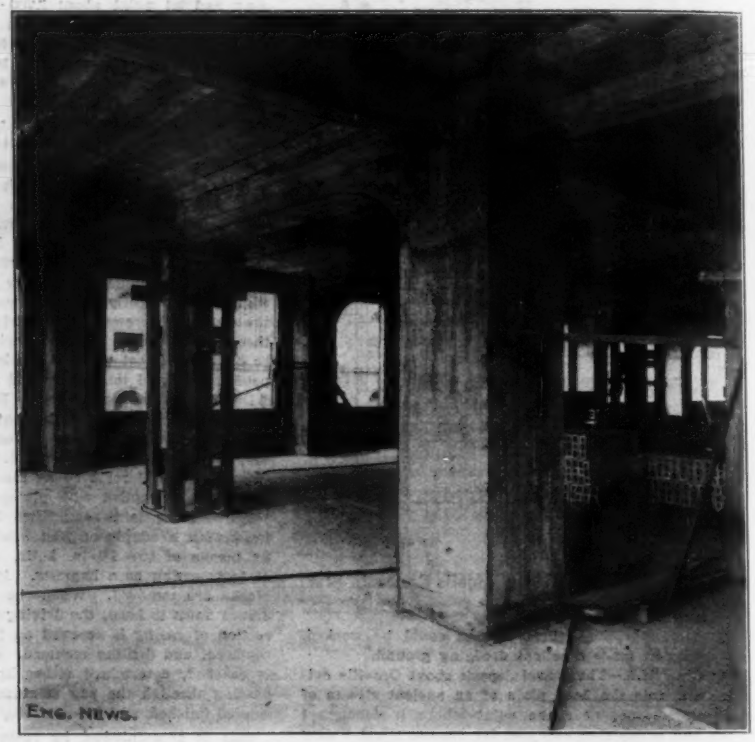


FIG. 17. VIEW OF COMPLETED FLOOR IN CONCRETE-STEEL BUILDING.

RULES OF AWARD FOR THE JOHN FRITZ MEDAL.

The John Fritz Medal Fund Corporation (Inc. April 18, 1903) has recently published a pamphlet setting forth the rules of award of the medal and the by-laws of the corporation. The board of award consists of 16 engineers from each of the four great engineering societies, as follows:

American Society of Civil Engineers—J. James R. Croes, Robert Moore, Alfred Noble, Chas. Warren Hunt.

American Institute of Mining Engineers—E. E. Olcott, E. G. Spilsbury, James Douglas, Charles Kirchhoff.

American Society of Mechanical Engineers—Gaetano Lanza, John E. Sweet, Robert W. Hunt, S. T. Wellman.

American Society of Electrical Engineers—Arthur E. Kennelly, Carl Hering, Charles P. Steinmetz, Charles F. Scott.

The officers of the board are: President, Alfred Noble; Secretary, Chas. Warren Hunt, 220 W. 57th St., New York; Treasurer, Charles Kirchhoff. The rules governing the award of the medal are substantially as follows:

(1) The John Fritz Medal was established by the professional associates and friends of John Fritz, of Bethlehem, Pennsylvania, U. S. A., on Aug. 21, 1902, his 80th birthday, to perpetuate the memory of his achievements in industrial progress.

(2) The medal shall be awarded for notable scientific or industrial achievement. There shall be no restriction on account of nationality or sex.

(3) The medal shall be of gold (9[illegible] —— ounces weight and —— fineness) and shall be accompanied by an engraved certificate.

(4) The medal may be awarded annually, but not oftener.

(5) No award of the medal shall be made to any one whose eligibility to the distinction has not been under consideration by the Board of Award for at least one year.

(6) Awards shall be made by a board of 16, appointed

FIG. 13. DEPOSITING CONCRETE FOR GIRDERS AND FLOORS IN THE INGALLS CONCRETE-STEEL BUILDING, CINCINNATI.

or chosen, in equal numbers from the membership of the four national societies (as already named).

(7) In case of failure of any of the national societies to make the original appointments as requested, the selection of representatives from its membership shall be made by those appointed from the other societies, and should any future vacancy occur by reason of the failure of any of the said societies to act, or otherwise, such vacancy shall be filled by the Board of Award, from the membership of the society so failing.

(8) Should one or more of the four national societies go out of existence its representation on the Board shall cease and terminate, and future awards shall be made by the representatives of the remaining societies.

(9) The vote of the Board of Award shall be by letter-ballot, which shall be canvassed not less than 30 days subsequent to its issue, and the affirmative vote of at least three-fourths of the Board of Award shall be necessary for an award.

The names and places of residence of the persons who shall be the directors of the corporation until its first annual meeting are as follows: [as above given.]

The corporation shall have the power to take, hold and receive and is hereby authorized and empowered to take, hold and receive, gifts of real and personal property by will or otherwise, for or in furtherance of the purposes and objects hereinbefore set forth.

The date for holding the annual meeting of the corporation shall be the third Friday in January in each year.

THE RAILWAY ACROSS THE ANDES from Guayaquil, Ecuador, to Quinto has reached Guamote, which is 126 miles from Guayaquil and 10,000 ft. above the sea. The most difficult part of the work is said to have been done, and only 24 miles remain to be built. Americans are building the road.

THE STRENGTH OF CONCRETE AS AFFECTED BY DIFFERENT PERCENTAGES OF WATER.*

By T. L. Doyle and E. R. Justice.

Laboratory tests on concrete mixtures show in general that comparatively dry mixtures well compacted by ramming develop greater strength in compression than wet mixtures. Nevertheless it is still an open question among engineers whether it is better to use dry or wet mixtures in practice. Many engineers hold that in view of greater cost of labor in laying dry mixtures of concrete and the difficulties of insuring a thoroughly solid mass even under the most rigid inspection, it is better in many cases to employ wetter mixtures. Again, some tests seem to indicate that wet mixtures of long-time tests show as great or greater strength than dry mixtures.

The object of this work was to determine experimentally the relation of strength between three mixtures, viz.: (1) dry; (2) medium wet; (3) wet. To describe more fully these mixtures it may be stated that the dry mixture was of a consistency of damp earth. The wet mixture was thoroughly saturated with water and when lightly rammed the water flushed freely to the surface. The medium mixture was of a consistency midway between these two.

Two kinds of cement, Alpha Portland and Hoffman natural, were used and two kinds of sand designated as white sand and bar sand, separate test blocks being made from each. The stone was 1-in. Birdsboro trap rock.

The voids in the stone were determined by filling a ⅛ cu. ft. measure with stone which had previously been soaked in water for 24 hours and then allowed to dry for 30 minutes before using. The measure was filled flush with stone and weighed, the voids were then filled with water and the mixture re-weighed—the difference being the weight of water added. This difference divided by $\frac{62.5}{8}$ gives the percentage of voids. This was repeated five times, giving a mean percentage of 47% voids. The voids in the sand were determined in the same manner and were for the white sand 45% and for the bar sand 44%.

It was found by trial that dry cement to the amount of 40% of the volume of sand could be worked into the sand. With a 1 to 2 mixture, the cement was slightly in excess. This when mixed with water was sufficient to fill the voids in five volumes of stone; but in the case of Portland cement the cubes were sometimes too strong to be broken on a testing machine of 100,000 lbs. capacity. A 1:3:5 mixture was therefore used in which the mortar was greatly in excess of the voids in the stone. With the natural cement, the proportion of 1:2:5 was adopted.

The mixtures used were a dry, a wet and a medium mixtures each, of Portland cement and white sand, Portland cement and bar sand, natural cement and white sand and natural cement and bar sand. Ten cubes were made of each mixture, five being allowed to stand for seven days and five for 28 days.

The dry sand and cement in the required proportions were placed on a board and thoroughly mixed; then water was added until the mortar so formed was of the consistency of damp earth. This amount of water was noted and from this the percentage of water in the dry mixture determined. The mortar was then thoroughly mixed with stone, which had been previously soaked and allowed to dry one-half hour before using. The aggregate was then rammed into 6-in. cubical iron molds by means of a wooden block and tamping bar.

The amount of water in the wet mixture was next determined. The sand and cement was mixed as before and water was added until the mortar assumed a very plastic state. The amount of water was noted and the percentage determined accordingly. For the medium mixture, the proportion of water is a mean between that of the dry and wet mixtures.

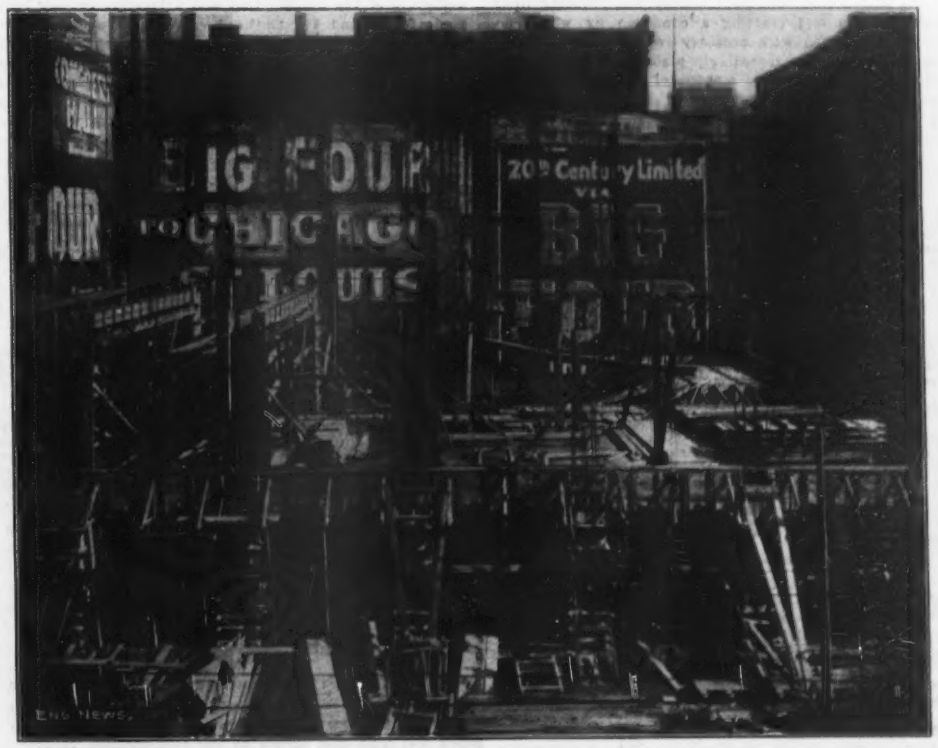


FIG. 14. INGALLS CONCRETE-STEEL OFFICE BUILDING DURING ERECTION.

The cubes were tested on a 100,000-lb. Olsen testing machine. They were placed between flat steel bearing surfaces with paper at top and botom to compensate for any slight irregularities of surface, thus tending to equalize the distribution of the pressure. The machine was run at a slow speed, 0.04-in. per minute. The average values obtained for the various cubes appear in the following table:

*Abstract of a thesis by T. L. Doyle and E. R. Justice, graduating students at the University of Pennsylvania.

Comparative Compression Strength in Pounds of 6-in. Concrete Cubes Mixed Wet, Medium Wet and Dry.

(These figures are the average of five tests in each case omitting results discarded on account of irregularity of cubes.)

	Portland Cement—Dry. 7 days.	Dry. 28 days.	Medium wet. 7 days.	Medium wet. 28 days.	Wet. 7 days.	Wet. 28 days.
Bar sand	47,800	92,250	44,287	68,100	44,820	47,550
White sand	59,325	84,775	53,800	88,970	52,070	55,260
Natural cement—						
Bar sand	9,280	17,320	10,540	18,240	11,820	12,050
White sand	15,460	25,425	9,220	16,900	4,975	10,160

For Portland cement, it will be seen that the strength usually decreases from the dry to the wet mixture. In the dry and medium mixtures, the difference between the seven and 28 days' tests is very marked, but with the wet mixture the difference is slight. This seems to show that the wet mixture reaches its final strength much more slowly. The bar sand gave slightly lower values than the white sand for like mixtures, showing that the cleaner, coarser and more angular sand was the better.

In the figures for natural cement is seen one notable exception to the general rule, namely, the mixture of bar sand, 7 days, which gave the lowest value for the dry mixture and the highest for the wet. The white sand again gave higher values in every case than the bar sand for the dry and medium mixtures. For the wet mixtures, the reverse is true for reasons unexplained. It is also noteworthy that the relative strength of the seven and 28-day tests was much less marked in the case of the bar sand (wet mixture) than for the other mixtures. The reason for this is not clear.

From the results, it is seen that in general, dry mixtures if well rammed are stronger than wet mixtures, provided the tests are made at the age of 28 days or less.

As previously stated, some tests are on record which indicate that if the wet mixtures are allowed to harden for long periods, they will ultimately be stronger than the dry mixtures.

TESTS OF PORTLAND CEMENT MORTAR EXPOSED TO COLD.*

By C. S. Gowen,† M. Am. Soc. C. E.

The following experiments were made with a view to getting some definite information on the effect of frost on Portland cement mortar, under the different conditions in which it may be desired to use it in cold weather. No facilities existed for maintaining a prolonged cold or uniform temperature and the briquettes were accordingly exposed to the open air, and so kept until it was evident that the tendency to "dry out" unduly was reducing their proper strength and creating a condition by which no basis of comparison with ordinary results could be had.

The briquettes were accordingly placed in water in July at the end of the first six months of the tests, and the author is inclined to the opinion that if this had been done earlier, at the end of the three months' tests, the 12 months' results would have showed much nearer the average 12 months' results of tests made in the ordinary way of briquettes kept in water continuously until broken. In the table given below each breaking weight given is the mean of eight briquettes broken, and it may be said that each set of breakings showed marked uniformity in the strength of the briquettes.

The results are from experiments made by the author while acting as Resident Engineer for the Aqueduct Commissioners of the city of New York, in charge of the New Croton Dam, and to them the author wishes to make proper acknowledgment for the use of these data.

EFFECT OF COLD UPON SETTING.

In the case of this lot of cement which was moderately quick setting (taking heavy wire in 63 mins. 2 to 1 briquettes and taking heavy wire in 71 mins. neat briquettes, under normal conditions of testing in laboratory) moderate cold, 22° and upward, delays setting but does not freeze. This is shown by the set of tests made for the intended temperature 24° to 32° of exposure.

The second set of tests (intended temperature of exposure 24° to 10°) show in the case of the 28th day and three months breakings (temperature 24° and 22°, respectively) a delayed setting which resulted in freezing during the night. The other breakings, 6, 9 and 12 months, show at lower temperatures, quick freezing.

The third set of tests (intended temperature of exposure 24° to 32°) was exposed to moderate cold after having taken heavy wire in laboratory.

The fourth set, a mixture with brine (intended temperature of exposure 20° to 10°) shows clearly the influence of the cold in delaying the set, as well as the effect of the brine in delaying freezing.

At temperature of exposure 16° +, the set occurred in 6 hours; 16° —, no set in 4 hours and no sign of freezing; 14° +, no set in 2¾ hours and no sign of freezing; 20° +, a set in 6½ hours with some indications of freezing; 26° —, no set in 2½ hours and no sign of freezing.

The fifth set of briquettes was exposed at a steady temperature of 18° ±, and all froze in 35 mins.

CONCLUSION.—A moderately quick setting cement can be used in temperatures about 20° without freezing with a 2:1 mixture.

The use of brine delays freezing, at least, at temperatures of about 15°, if it does not wholly prevent it before the set has occurred.

EFFECT OF COLD UPON BREAKING STRENGTH.

It is apparent that the general falling off at the end of six months is due to air exposure, the rise for 9 and 12 months after being placed in water being marked, and the author is of the opinion that had briquettes enough been made for 15 and 18 months' breakings there would have been a uniform increase in strength, comparing favorably with general results from laboratory tests; a summary of which has been added to the tabular statements. The six months' breakings of the various sets show a much greater uniformity than those of one month and three months, as might have been expected, the extremes being 287 and 366 lbs.

At 9 months sets 1 to 4 agree closely.
sets 3 to 5 agree closely.

While set 2 is lower in its breaking weight than either of the others.

At 12 months sets 2 to 4 agree closely.
sets 3 to 5 agree closely.

While set 1 comes between these extremes which vary between 510 and 602 lbs., an extreme variation, not much greater than indicated by the six months' breakings; and much less than that shown by the nine months' results.

CONCLUSION.—The general result is favorable to the use of brine at low temperatures; also there is no indication that freezing reduces the ultimate strength of the mortar, although it delays the action of setting.

In this particular example the frozen set No. 2 shows better at 12 months than frozen set No. 5, but not so well at 9 months, where the relative difference is the other way.

At 12 months sets Nos. 2 and 4 ("frozen at low temperature" and "brine") agree closely.

Set No. 1 comes next ("mixed at moderate temperature"), and sets Nos. 3 and 5 follow.

There seems to be nothing in the results shown in No. 3 to indicate an advantage in securing a set before exposure to freezing temperature.

The above results are relative rather than conclusive as it is impossible to say what would have been the results at the end of the year, and how they would have compared with the general average given for briquettes tested under normal conditions if they had not been exposed to the varying temperatures of spring and early summer and to "drying out."

These briquettes were mixed in February, 1897, as opportunity and the required temperatures occurred, and the records of the time of setting were made as carefully as was practicable under the circumstances.

The temperatures of the air at the time of the final test for the set were not taken, but as a rule the temperature rose or fell, as indicated, steadily during the time that elapsed while the observation was made. These results are submitted for what they may be worth, as the author does not know of any series of tests extending over so long a time and at the same time covering such extremes and variations of temperature.

The following, showing the results obtained by tests made under ordinary laboratory conditions, when brine was used, are added here, and the conclusion seems to be plain that the effect of brine is to delay setting temporarily, while not affecting the ultimate strength of the mortar materially.

Giant Portland, 2 to 1 briquettes.

Per cent. of water used to weight of cement, 40.

Time to take heavy wire, fresh water briquettes, 241 mins.; salt water briquettes, 306 mins.

	1wk.	1m.	3mos.	6mos.	9mos.	12mos.
Fresh water used	236	289	414	549	554	572
Salt water used	126	231	294	424	452	576

Giant Portland 3 to 1 briquettes.

Per cent. of water used to weight of cement, 50.

Time to take heavy wire, fresh water briquettes, 350 mins.; salt water briquettes, 407 mins.

	1wk.	1m.	3mos.	6mos.	9mos.	12mos.
Fresh water used	112	183	268	335	351	408
Salt water used	68	131	215	266	301	413

Etandard sand used (crushed quartz).

The brine used was strong enough to float a potato, about a 10% solution by weight.

Each of the above results is the mean of ten breakings, in pounds per square inch.

The briquettes were placed in air 24 hours, and then immersed in water until broken.

BREAKING WEIGHTS OF 2:1 MORTAR BRIQUETTES, LBS., PER SQ. IN., EXPOSED TO COLD, AT NEW CROTON DAM.
Cement used Giant Portland, sand used crushed quartz—Lot 209, 1,476 bbls.. Each breaking weight given is the mean of eight breakings.

Temperature Intended.	28 days. Br'k'g w't., lbs. per sq.in.	28 days. Temp. exposure, degs.	28 days. Time to take heavy wire.	3 months. Br'k'g w't., lbs. per sq.in.	3 months. Temp. exposure, degs.	3 months. Time to take heavy wire.	6 months. Br'k'g wt., lbs. per sq.in.	6 months. Temp. exposure, degs.	6 months. Time to take heavy wire.	9 months. Br'k'g w't., lbs. per sq.in.	9 months. Temp. exposure, degs.	9 months. Time to take heavy wire.	12 months Br'k'g wt., lbs. per sq.in.	12 months Temp. exposure, degs.	12 months Time to take heavy wire.
24 to 32°	370	22 r.	4 hrs.†	474	27 r.	4½ hrs.†	366	34 f.	.....	553	28 r.	4½ hrs.†	553	26 r.	7 hrs. s.
24 " 10°	458*	24 f.	Night.	455	22 s.	Night.	347	12 r.	15 mins.‡	381	14 r.	15 mins.‡	586	16 r.	45 mins.§
24 " 32°	371	28 f.	65 mins.	413	28 f.	65 mins.	314	28 f.	65 mins.	452	28 f.	65 mins.	510	28 f.	45 mins.
20 " 10°	272	16 r.	6 hrs.	360	16 f.	4 hrs. r.‖	287	14 r.	2¾ hrs. r.+‖	567	20 r.	5½ hrs.‡	602	26 f.	2½ hrs.‡‖
20 " 10°	255	18 s.	35 mins.	246	18 s.	35 mins.‡	300	18 s.	35 mins.‡	437	18 s.	35 mins.‡	512	16 s.	35 mins.‡

*This set was broken on a day when the temperature was 16°; a ninth briquette was thoroughly thawed on same day and broke at 210 lbs.
†Did not appear frozen when it took heavy wire. ‡Frozen at end of time noted, and took wire. §Froze slowly and took heavy wire.
‖Had not set at end of time noted. ‡Some signs shown of freezing. +One briquette made with fresh water froze and took heavy wire in 20 mins.

Remarks: 24 to 32°.—Placed in cold air at temperature noted immediately after mixing; fresh water used. 24 to 10°.—Placed in cold air at temperature noted immediately after mixing; fresh water used. 20 to 10°.—Placed in cold air at temperature noted immediately after mixing; fresh water used. In column of "temperature of exposure" (r.) indicates a rising temperature; (f.) a falling temperature, and (s.) a steady temperature. All briquettes were left in open air in a dry but not sunny place until the 3 months' break was made (about April 15); they were put in a damp place until the 6 months' break was made (about July 15), and then they were placed in water until finally broken. The brine used was a solution strong enough to float a potato, about 10% by weight of salt to weight of water.

Average Breaking Weights of 2:1 Mortar Briquettes Giant Portland Cement, Broken at New Croton Dam in 1896, 1897, 1898.

		Lot 209, 1,476 bbls.
Time	28 days.	...
No. of breakings	690	10
Avg breaking weight, lbs. per sq.in.	441	488
Time	3 mos.	...*
No. of breakings	215	...
Avg breaking weight, lbs. per sq. in.	563	...
Time	6 mos.	...
No. of breakings	185	...
Avg breaking weight, lbs. jer sq. in.	657	...
Time	9 mos.	...
No. of breakings	155	...
Avg. breaking weight, lbs. per sq. in.	671	...
Time	12 mos.	...
No. of breakings	165	...
Avg. breaking weight, lbs. per sq. in.	663	...

*Normal test of this lot not continued after 28 days.

Time of taking heavy wire, mean of 70 tests (2—1) briquettes, 63 mins.; mean of 70 tests, neat briquettes, 71 mins; average breaking weight, mean of 70 tests (2—1), briquettes, 1 week, 344 lbs.

*A paper read before the Cement Section of the American Society for Testing Materials, July 3, 1903.
†Engineer, New Croton Dam, Ossining, N. Y.

THE RETIREMENT OF REAR ADMIRAL GEORGE W. MELVILLE, ENGINEER IN CHIEF, UNITED STATES NAVY.

By W. M. McFarland,* M. Am. Soc. M. E.

The retirement on Aug. 8, next, of Rear Admiral George W. Melville, Engineer in Chief, U. S. Navy, and Chief of the Bureau of Steam Engineering, will remove from active public service a remarkable and heroic figure. Admiral Melville will have completed exactly sixteen years of continuous service as the head of this great constructive Engineering Bureau, a period which, if not the longest in the entire history of the Navy Department, is certainly the longest for many years. Admiral Melville's work in the upbuilding of the new navy has brought him so constantly before the engineering public that your readers are probably fairly well posted as to the work he has done, but such exceptional service as his is entitled to more than a passing notice when the time comes for him to lay down the office which he has filled with such great credit to himself personally, and to the engineering profession, and with such benefit to the government.

*Acting Vice-President Westinghouse Electric & Manufacturing Co., Pittsburg, Pa.; formerly Passed Assistant Engineer, U. S. Navy.

Admiral Melville was appointed Engineer in Chief, Aug. 8, 1887, while the Hon. William C. Whitney was Secretary of the Navy. It was distinctly an appointment on account of merit, for if Melville had any politics they were rather Republican than Democratic. His reputation as one of the most skillful engineers in the navy was already established, added to which was his magnificent Arctic record in the cruise of the "Jeannette," where he had command of the only one of the boats whose crew was finally saved. He was known as a man to be selected for difficult and dangerous work, and had shown that marked executive ability which has always been one of his most striking traits of character.

When he came into office the four Roach cruisers, which were the beginning of the new navy, were in commission and contracts had been placed for a few of the succeeding vessels. Except these few, however, all of the machinery of our navy as it is to-day has been designed and built under his supervision and responsibility, the aggregate horse-power amounting to just about one million. This, of course, does not represent the total amount of designing done by the bureau under his direction, as in many cases alternative designs and sketch plans have been prepared which subsequent changes in the general features of the vessel caused to be given up.

It is very interesting to note at the end of his term that Admiral Melville was the first Engineer in Chief of any navy to come out distinctly and strongly as an advocate of water-tube boilers, as a result of which the coast-defense vessel "Monterey" was fitted with Ward boilers of about 5,000 HP. He also adopted water-tube boilers exclusively for steam launches, with the best results, a practice which has been uniformly followed. The Admiral did not, however, because he was the first to advocate water-tube boilers for naval vessels, allow himself to be carried away with the idea, but felt that the best interests of the navy required that the first ones installed should be tested thoroughly before launching out into this form of boiler as a settled policy. The benefit of this line of action is shown by comparison with other navies, which, adopting water-tube boilers at a later date, pushed the matter hastily and with results which in many cases were very unfortunate.

The Admiral has kept our Navy ever in the forefront of progress. Without attempting to go in detail into all of his improvements, mention may be made of the two triple-screw flyers "Columbia" and "Minneapolis," which were the first very large vessels to use this system, each developing about 20,000 HP. on trial and breaking the speed record for naval vessels larger than torpedo boats at the time. Foreign navies, except that of England, have followed this lead and use triple-screw vessels very largely, but for some reason, which has never been clear to laymen, the Admiral's desire to continue the policy of using triple-screws has been negatived by the Navy Department. The important point is the courage and foresight shown by him in using triple screws at the time he did. He has also directed numerous experiments on the use of oil fuel, the latest of which are barely concluded.

Mention may also be made of the Admiral's consistent fight for the highest possible speeds in our vessels of all types, in which, unfortunately, he has not always had the support of his colleagues. Back in 1898 the Admiral had to fight single handed for high-speed battleships, in which, it is a pleasure to say, he had the cordial endorsement of the technical press. At that time, when all other nations were building battleships of at least 18 knots, it was seriously proposed by the other authorities in the Navy Department to copy our slower vessels of 16 knots, but the Admiral's determined stand, backed by the technical press and by the willingness of the contractors to furnish such ships, enabled us to get vessels of 18 knots.

Turning to another aspect of his career, it is to be noted that the Engineer in Chief of the Navy is not, as might be supposed by those unfamiliar with naval organization, a technical officer pure and simple, but the head of the Engineer Corps of the Navy, and as such charged with the administration of its personnel. Your readers are familiar with the long fight which the engineers in the Navy had made, lasting more than fifty years, for a tardy recognition of their official status. Up to 1899 engineer officers were considered staff officers, and as such were given what was called relative rank and with professional titles only. Inasmuch as their work has come to be very largely executive, involving the command of a considerable portion of the ship's company, the engineers had felt that, like their brothers in the Army, they should have a military title and actual rank to enable them to perform their duties efficiently. This was opposed for years by the executive or line officers of the Navy, but the splendid work done by the Engineer Corps under Melville's direction and his own tremendous personality gradually brought about a better feeling on the part of the line officers, until in 1897 the Personnel Board was appointed by Secretary Long to consider these questions of rank and was presided over by President Roosevelt, then Assistant Secretary of the Navy. The result of the deliberations of this Board, largely due to the tact and grasp of naval affairs possessed by Mr. Roosevelt, was a tremendous reorganization of the entire personnel of the Navy, which consolidated the engineers with the line, making them what they properly are, executive officers, and aiming to make the future naval officer primarily an engineer. Doubtless this result would not have been reached for many years had it not been for Admiral Melville's splendid personality and magnificent work in the administration of his office. At the beginning of this year the British Navy followed the lead set by our own, and has arranged for all of their executive officers to be trained for duty both on deck and in the engine room. While there are differences in details as compared with our own system, it nevertheless is essentially the same and was undoubtedly brought about by the change we had made.

One of the remarkable characteristics of Admiral Melville, and one which has enabled him to accomplish the immense amount of work he has done, has been his ability to inspire admiration and respect in the officers who have been his subordinates and assistants. There probably has never been an instance where the head of an office possessed so completely the confidence and loyal affection of all those under him. An officer who was for many years one of his assistants has said that after a long intimacy with the Admiral in which he had learned to admire his heroism, courage, and many other noble qualities, it seemed to him that this inspiration of loyalty in his assistants was the most remarkable thing about this remarkable man. It is easy for an outsider to see one of the things which contributed to this, because there never has been the head of a great office who was more ready to give credit to his assistants for the work done by them and the suggestions which they have made.

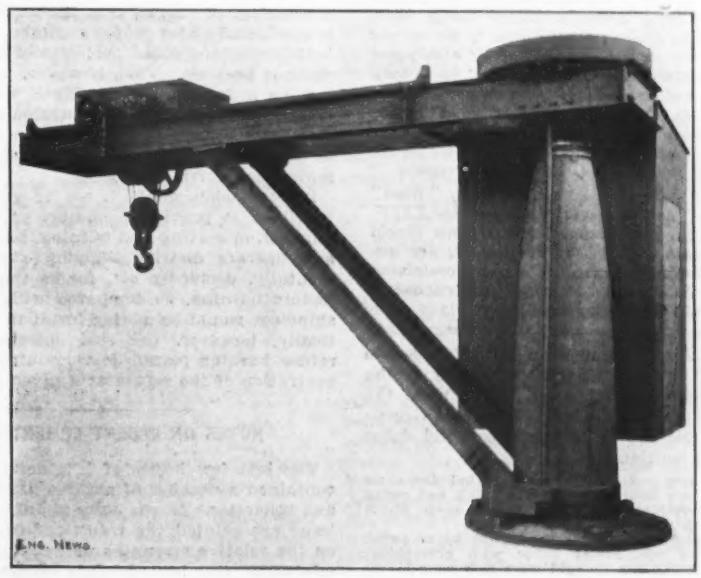
Such cases as Admiral Melville's are the exceptions which call attention to the wisdom of the rule of retiring officers from active duty when they pass the age of 62. He is only a few months past that age and, in spite of his venerable appearance, is an active and vigorous both mentally and physically as most men at fifty. It is a great misfortune that this rule should deprive the Government of his services, but inasmuch as it has recently become customary to call on retired officers of such great ability as his for special service, we have no doubt that the Government will not lose the benefit of his talents.

There is reason to believe that engineers generally are coming to realize much more fully than they did once that such a record as that of Admiral Melville is not merely a personal one, but that it sheds luster on the engineering profession. Even those who are not engineers ought to feel proud of an American citizen who has devoted all his life to the Government service, and whose magnificent work forms a monument to be seen by all men. But as engineers we can take special pride that this work is done by one of our own family, so that we as engineers share in the luster of his renown.

ELECTRIC PILLAR CRANES FOR HANDLING CUPOLA CHARGES.

The three cupolas in the foundry of the new shops of the Allis-Chalmers Co., at West Allis, Wis., are charged by cars running on inclined tracks, these cars being operated by hydraulic power. In the yard at the foot of the inclines, the skips or cars are handled by electric pillar cranes, which pick them off the track, swing them into convenient position for loading, and then replace them on the track.

One of these cranes is shown in the accompanying illustration. It has a heavy pillar revolving on the base, and the horizontal jib is supported by the struts. The effective radius of the jib is 16 ft., and the hoisting capacity is four tons. The operator controls all the motions from a small cabin which swings with the crane. There are three direct-current 220-volt motors for swinging the crane, racking the trolley in and out, and hoisting the load. The speeds are 20 to 50 ft. per minute for hoisting, 100 to 150 ft. per minute for racking the trolley, and three to five revolutions per minute for swinging.



ELECTRIC PILLAR CRANE FOR HANDLING FOUNDRY CUPOLA CHARGES AT THE WEST ALLIS SHOPS OF THE ALLIS-CHALMERS CO.

Northern Engineering Works, Detroit, Mich., Builders.

These cranes were built by the Northern Engineering Works, of Detroit, Mich., and we are indebted to the builders for photographs and other information.

THE BATTLESHIP SPEED RECORD ACROSS THE Atlantic was again broken when the U. S. battleship "Kearsarge" reached Bar Harbor, on July 26, from Portsmouth, England, in 9 d. 4 h. 15 m. The distance was about 2,900 miles and the average speed 13.12 knots against heavy head winds and seas; she consumed 1,178 tons of coal on the trip. The same ship made the run out, from Tompkinsville, New York Bay, to the Needles, England, in 10 d. 16 h., the average speed being a little over 12 knots.

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ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.

Under the title "No Dam Can Be Safe," a fine bit of illogical writing recently appeared in the "News," of Ithaca, N. Y. After asking what the people of Johnstown, Oakford Park, and Greenville, S. C., would say to attempts to build dams above them it continues:

> These people know and realize too well that dams, no matter how they are built, are likely to break and sweep to destruction everything in the way of the water which they may store up.
>
> Yesterday the city of Baltimore was visited by an awful storm. Wind and rain raised havoc with everything. Buildings which every one thought were safe were blown down; roofs which were built to withstand all storms were torn from houses like so much tissue paper.
>
> Scarcely a day passes but the press dispatches tell us of some great destruction of life and property by the forces of nature, which human hands and the greatest engineers cannot master.
>
> No dam can be built that will be safe. Then why, for Heaven's sake, is it necessary to jeopardize the life and property of Ithacans by building a big reservoir in Six Mile creek

If buildings and dams alike cannot be built to withstand the forces of nature, why stop building one and not the other?

The people of Winnetka, Ill., have shown their belief in municipal ownership by establishing water-works and electric lights, and by voting $75,000 for the erection of gas works. They, or their city council, have also "passed a resolution declaring municipal ownership of public utilities to be a success." Commenting on these facts the Chicago "Tribune" says:

> They must not suppose, however, that in large cities municipal ownership can be introduced as successfully as in a small city like Winnetka. The people of Winnetka have three advantages. First, they are few enough to have strong local patriotism and large local information. Second, they are wealthy enough to be open to no petty boodle temptations. Third, they are intelligent enough to be able to understand and follow their municipal enterprises and to keep politics out of them.
>
> The citizens of larger cities are not blessed with these advantages. Their progress toward municipal ownership will have to be slower.

Truthful and suggestive as these statements are in their relation to municipal ownership, it should be remembered that local patriotism, freedom from boodle temptations, intelligence, and the elimination of politics of the baser sort are essential to good municipal government, and that the latter is of far more importance than any question of municipal or private ownership. Such ideals, however slow and difficult of attainment, are a possibility in large as well as small cities.

The greater part of the expense of garbage collection and disposal in many cities is due to transportation charges. Where, as is so frequently the case, there is a long haul by land why are not the street railways more often pressed into service?

The Savannah Electric Co., of Savannah, Ga., has just received three special garbage cars for this purpose, and a recent contract for the removal of refuse in Brooklyn Borough, New York city, included a plan for 13 loading stations and many electric railway cars.

Many as are the attractions of such a plan, there are also many drawbacks. Among these are: (1) A more or less expensive equipment which, under the exigencies of garbage and refuse contracts in most cities, are liable to be thrown out of use in from one to three or possibly five years; (2) objections of property-owners to the location of the necessary loading stations near their premises; (3) opposition to hauling refuse on the same tracks as are used for passenger traffic; (4) objections to locating disposal plants, and still greater objections to locating dumps, near trolley lines. As to these drawbacks the first can be counteracted largely by longer contracts, and wholly or almost wholly by municipal collection and disposal. The second and third are largely based on sentiment, asssuming, of course, that the service in question is conducted under proper sanitary conditions, and both objections could be lessened wherever night work is possible. This, however, would generally involve a day storage of refuse, which would increase the objection to the location of the shipping stations. Branch railway lines to disposal works at more or less remote points would lessen or remove the fourth difficulty.

In conclusion, might it not be quite as feasible, in cities, to provide a number of district refuse burning, or sorting and burning, plants as to erect and operate district shipping stations? Sentimentally, decidedly not, for in the minds of objectors burning, as compared with receiving and shipping, would be adding insult to injury. Practically, however, the real objections to proper refuse burning plants lie very largely in the concentration of the refuse at a given center.

NOTES ON RECENT CEMENT TESTS.

The last few issues of Engineering News have contained a number of notable papers on cements and concretes. In our issue of July 16 and in this issue are printed the results of two sets of tests on the relative strengths of wet and dry concrete mixtures. Our issue of July 23 contained a most interesting paper on soundness tests of Portland cement, and in this issue are described tests to determine the effects of exposure to cold on Portland cement mortars. All four of these investigations relate to questions of the first importance in the reception and employment of hydraulic cements in engineering works, and they deserve to be read with attention by cement users.

The papers on the relative strengths of wet and dry concrete mixtures contained in this issue and our issue of July 16 are both by graduating students in well-known engineering schools, and these young men deserve commendation for their endeavor to present some definite information on a much disputed question. Unfortunately, however, their work has resulted in contradictory information and the whole question is left in much the same status as it was before their labors. The tests described in our issue of July 16 were overwhelmingly in favor of wet mixtures as giving a better and stronger concrete than dry mixtures. Those described in this issue are nearly as decisive in favor of the opposite conclusion. With the two exceptions of the rather crude tests made by Mr. Irving Hitz and Mr. H. W. Parkhurst, and described in the Journal of the Western Society of Engineers for October, 1900, and June, 1902, these are the only comparative tests of wet and dry concrete mixtures which are available to engineers. Mr. Hitz and Mr. Parkhurst both molded large cubes of wet and dry concrete and split them by wedges, and from this method of testing Mr. Hitz concluded that a moderate excess of water did not injure the strength of concrete, and was the only means by which a compact concrete could be obtained. Mr. Parkhurst's conclusions from his experiment were that a medium wet mixture gave a stronger concrete than either a dry or a wet mixture. Evidently the results of the tests before us are far from settling the much mooted question whether a wet mixture or a dry mixture makes the stronger concrete, and there is still an opportunity for some of our college laboratories to perform a valuable service to engineers by making a series of tests which will decide this question.

Although the tests so far made to determine the relative strengths of dry and wet concrete mixtures leave the matter very much in doubt, it is not likely that this will have any great influence in retarding the increasing growth of wet mixtures in the favor of engineers. In some respects this growth has been a remarkable one. It is only a few years ago that the proposition to use for the important hydraulic works of the Chicago Drainage Canal a concrete which would quake on being rammed was met by such a volley of protests that the engineers of that work were compelled to defend their action before the public. To-day quaking mixtures are quite as commonly employed, probably, as dry ones, and there are large engineering works under way in which the concrete being used is almost a liquid mixture compared with the standard of consistency set forth by the text books and generally adopted in practice ten years ago. The principal reason for these changed conditions is that less labor is required and a more compact concrete is secured with the same amount of labor when wet mixtures are employed, and that in reinforced concrete construction, particularly when the reinforcement is composed of small elements, the only surety the engineer has that his steel is thoroughly in contact with the concrete is obtained by using a very wet mixture. It is safe to say that these advantages will be sufficient to offset any slight decrease in strength due to the use of wet mixtures even were it firmly established that there was such a decrease.

The paper by Mr. C. S. Gowen, M. Am. Soc. C. E., describing the tests made by him to determine the effect of exposure to cold upon the setting of cement mortar is published nearly in full elsewhere in this issue, and its conclusions are so concisely expressed that they need little comment. Briefly stated, Mr. Gowen concludes that a 2 to 1 mortar made of moderately quick-setting cement can be used at temperatures as low as 20° F. without freezing; that at a temperature of 15° F. the addition of brine considerably delays freezing if it does not absolutely prevent it before set takes place; and that neither the addition of brine nor freezing reduces the ultimate strength of mortar, although freezing delays the action of setting. There is less fear now than formerly of using cement in freezing weather, but there are probably few engineers who would do it if it were avoidable. Mr. Gowen's tests can hardly be said to have shown this precaution unnecessary, but they will relieve the mind of the engineer who has to use mortar in freezing weather from some of the nervousness with which he has in the past watched developments in cement structures built in cold weather.

There is probably no single test for cements which has come in for so much vigorous condemnation as the so-called accelerated test for soundness. In many respects this condemnation is fully warranted. Some of the forms of accelerated tests which have been advocated are chiefly notable for the ingenuity of the torture which they require the cement to undergo, and few, if any, of the proposed tests have failed to suffer from the too sweeping claims made by their advocates. Cement manufacturers quite generally condemn accelerated tests. They point out that they are widely different in character and severity; that the size and form of test piece differ greatly; that the duration of the heat treatment varies from a couple of hours to as many days; and that the age of the test piece required is not uniform. Their chief argument, however, has been that cements which endure the heat test often disintegrate after being employed in construction while cements which have stood perfectly in construction have been classed as unsound by the accelerated tests.

In a paper read before the cement section of the American Society for Testing Materials, and published in our issue of July 23, Mr. W. P. Taylor reviews these various objections to accelerated

tests and determines their real importance from results obtained in the municipal testing laboratories at Philadelphia. We believe that this paper will prove a strong argument for the more general use of accelerated tests, and that it should be carefully studied by every large cement user. So far as the boiling test is concerned, Mr. Taylor effectually disposes of the argument that the accelerated test can be dismissed from the consideration of engineers merely because there is a considerable measure of truth and justness in the criticisms outlined. The accelerated test by boiling water is by no means infallible, but neither are some of the other and more common tests to which cements are subjected. In any case a test which in 96 instances out of every hundred proves to be an accurate determination of so important a fault of a cement as unsoundness is well worthy of careful consideration, even though it failed in Mr. Taylor's tests to disclose unsoundness in the case of 0.5% of the unsound cements, and in a considerable number of cases indicated unsoundness in cements which stood well in construction. The explanation of this last result is found by Mr. Taylor in the fact that cements usually have some time between the test and the time at which they are used in which to season and render the expansive elements ineffective and in the fact that the disintegration action of a cement is always greater when mixed neat than when made into a mixture with twice or three times its volume of sand.

Mr. Taylor finds that within reasonable limits no such curse is attached to irregularities in form, size and age of test piece and in duration of the heat treatment as some would make engineers believe. To be more specific he finds that the form of the test piece has little or no influence on the results, whether it be a cake, a ball, a wedge or a pat, if the duration of the test be sufficient; that generally a cement which will endure heat treatment for three or four hours will endure it for a much longer time, and that if the test pieces be given time to develop full set the age afterwards at which they are tested cuts but little figure. These are very comforting assertions to those who have long sought for a recognition of the merits of the accelerated test, and they are further useful in indicating that no great obstacle stands in the path of doing away with many of the vagaries in shape, size and age of test pieces and duration of test which now characterize engineers' specifications for heat tests. It is hardly needful to say that greater uniformity is desirable if it can be brought about and within a short time we trust to see it accomplished.

One of the greatest obstacles to the general use of accelerated tests lies in the fact that a cement when fresh or when mixed neat will show unsoundness which it will not show after a period of seasoning or when mixed with sand. To condemn such a cement by a heat test made on fresh, neat material is of course unjust to the manufacturer and the purchaser, but it obviously cannot be allowed to pass unchallenged if the heat test is to have any reason for existing in the specifications. Probably the only practicable course to take in such instances is to hold the cement for further tests, giving it a chance to age and furnish the tester an opportunity to observe the results of the normal cold water tests and verify his previous observations.

LETTERS TO THE EDITOR.

Medical Outfits for Camping Parties.

Sir: I have noticed the various suggestions you have recently published as to suitable medical outfits for camping parties. It therefore occurs to me to suggest that the hints on medicine and surgery and the make-up of a medicine chest published in my "Topographic Surveying," p. 861, and preceding, may be of interest to your readers.

Those notes were prepared from experience and with the advice of a physician of reputation. They were proof-read by another, a surgeon of experience.

Yours truly, H. M. Wilson,
U. S. Geological Survey.

Washington, D. C., July 8, 1903.

A Heavy Rainfall at Baltimore, Md.

Sir: I note in your issue of July 16, a report from the Commissioner of Public Works for the Borough of Richmond, New York city, of an unusual rainfall which occurred there on Monday, June 29.

We had in Baltimore, on July 12, 1903, a rainfall far in excess of the above report. Upon that day rain fell here for five minutes at the rate of 9.6 ins. per hour. In 35 minutes, 2.87 ins. fell, which is at the rate of 4.92 ins. per hour.

There follows a record of the rain for each five minutes, also a record of the greatest rainfalls previously observed by the Weather Bureau at this place since the organization of the bureau.

Official Report of the U. S. Weather Bureau.

Rainfall of July 12, 1903.	Actual.	Rate per hour, ins.	Largest rate per hour previously observed.
For 5 mins.,	0.80	9.60	6.84 ins.
" 10 "	1.35	8.10	4.80 "
" 15 "	1.87	7.48	4.00 "
" 20 "	2.22	6.66	.... "
" 25 "	2.46	5.90	4.25 "
" 30 "	2.68	5.36	2.64 "
" 35 "	2.87	4.92	1.89 "

Yours very truly, B. T. Fendall, City Engineer.

Baltimore, Md., July 20, 1903.

The Heaviest Recorded Rainfall at Atlantic City, N. J.

Sir: The storm which passed over Atlantic City on July 22 was accompanied by the heaviest precipitation on record by the Weather Bureau at this station. This was:

From 5.30 p. m. to 8 p. m.	4.85 ins.
From 8.00 p. m. to midnight	.60 "
Total in 4 hrs. 30 mins.	5.45 "

At our pumping station, about six miles distant, the precipitation up to 8 p. m. was but 1.80 ins., and from 8 p. m. to 8 a. m., July 23, .33 in.; total, 2.13 ins.

Reports had been received by the Weather Bureau yesterday noon from other New Jersey stations, in none of which the precipitation exceeded ½-in., indicating the heavy rainfall to be largely local, although extending over five or six hours. The storm was accompanied throughout by a vivid display of lightning.

The rapidity with which this great volume of water was taken up by the prevailing sandy soil was remarkable, as, with the exception of wet basements and a few isolated pools, there was no indication of the recent downpour the next morning. Yours truly,

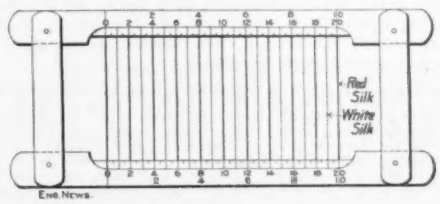
Kenneth Allen,
Engr. and Supt. Water Department.

Atlantic City, N. J., July 24, 1903.

Correct Figures for Urban Transportation in New York City.

Sir: In your issue of July 9 you state that "the urban railway transportation in New York city for the past year aggregated 410,414,064 passengers on the elevated lines and surface street-car lines in all the boroughs of the city." This figure is clearly in error, the total number of passengers carried annually being vastly greater, amounting to nearly one thousand millions, not including transfers. In the recent report of the State Board of Railroad Commissioners, "In the Matter of the Transportation Problem in Greater New York," dated June 30, 1903, the same figure occurs which you gave, and its tabular caption makes it appear as if for the entire year 1903. But a close examination of preceding tables in the same report, which seem to have been very hastily prepared, shows plainly that it is for the four months of February, March, April and May of 1903, and that the comparison is with the corresponding period of 1902. On the assumption that the travel for these four months is typical for the entire year, the total annual urban transportation obviously would be 1,231,242,192, including transfers. From the same report it may be found, with some trouble, that the number of transfers issued during the four months was 78,023,778, which at the same rate would indicate 234,071,334 for the year. This would make the yearly number of actual passengers 997,170,858. Allowing for the normal increase of travel during the remaining months of the year, the reasonable estimate of urban travel in Greater New York for 1903 will exceed 1,000,000,000. M. A. C.

New York City, July 24, 1903.



Homemade Instrument for Interpolating Contours.
I. T. Farnham, West Newton, Mass.

Prior Use of the Parallelogram for Interpolating Contours.

Sir: I note in your issue of July 16 a description of an instrument for interpolating contours. This was interesting to me as it recalled to my mind an apparatus on exactly the same principle which I devised and constructed some five years ago to accomplish the laborious process of interpolating contours from points located on plan by an extensive stadia survey made in Newton. A search among my discarded drafting instruments brought forth my original device which is so simple as to be easily constructed in any office. A description may be of interest to your readers.

The parallelogram was made of thin strips of wood about one-fifth of an inch thick, taken from a cigar box. These were cut to the desired shape, finished with sand paper, and pinned together at the corners with ordinary pins inserted so that the heads would lay on the plan, the points being cut off flush with the top or end strip of the parallelogram. Care was taken to have the pins exactly on the line of the inner edge of the side strip, as this edge was the line of attachment for the threads. A small cut made by the knife across the underside of the side strip perpendicular to the edge served as a groove to attach the threads which were held in place by a little glue. By using thread of two colors alternating and marking the scale with two sets of figures, it was possible to cover double the range of contour distances with a given length of scale. This is illustrated in the accompanying sketch. Very truly yours,

Irving T. Farnham, City Engineer.

West Newton, Mass., July 21, 1903.

(For the sake of simplicity we omitted to state in our article that the instrument devised by Mr. Kubala had two rows of figures on each scale, one row having double the spacing of the other, just as in Mr. Farnham's device. It will be seen, then, that Mr. Kubala's instrument is identical in every respect with that previously constructed by Mr. Farnham, except that the latter used two colors of thread—an advantageous feature. As this is without doubt the best device for interpolating contours that has yet been proposed or used, we feel sure that every engineer having topographic mapping to do will make one for himself, unless the idea is taken up by instrument makers. In the latter case it would be of advantage to make the frame of transparent material, as celluloid, which would take the few graduations necessary without requiring overlaid scale-strips.

It is worth while to again point out, apropos of Mr. Farnham's letter, that if an engineer has a special instrument or device of his own he should make it known to his professional brethren, which he can do most conveniently, perhaps, by a letter to Engineering News.—Ed.)

A Union Engineering Building Held to be of no Benefit Unless the National Engineering Societies Combine.

Sir: Notwithstanding your able and persuasive arguments in favor of a united engineering building, I still remain unconvinced that the profession of engineering will be benefited by such a union, unless the result is to be a National Engineering Society, embodying all branches of the profession; the basis of which will be our present American Society of Civil Engineers. For many reasons this should be the national society, principally because it already embodies the most distinguished members of all branches.

Having a circular before me from the Secretary of the Canadian Society of Civil Engineers, I read: "The Council, in accordance with the terms of the resolution, has decided to form four sections of engineering in the Society, viz., Electrical, Mechanical, Mining and General." Further, "It has recommended that in future four vice-presidents be elected, instead of three, and that they act as presidents of the several sections," and, again, "that the papers read and discussed by each section be published in the Society Transactions, subject to the approval of the Committee on Papers," and "that the council has appointed the following officers for the present year," here giving the names of the officers.

The resolution referred to, and adopted at the last annual meeting of the Canadian Society, reads: "That the matter of forming sections in the society to specialize in the several departments in engineering be referred to the Council with power to act."

Mr. Editor, I feel strongly that membership in one great National Society should be the goal of every young graduate, whether his technical course has been in Electrical, Mechanical, Mining or General Engineering. His specialty, of course, would lead him to enroll himself in one or other of the "sections," and would be the proper place in which to read and discuss his technical papers, until they were of such importance as to be selected by the council, or board of direction, for presentation before the national society.

Any one looking over the papers presented at the recent conference of the Institution of Civil Engineers in London cannot fail but be struck with the wide range of subjects treated therein, electrical, mechanical and general.

The Institution is evidently, in the opinion of the engineering profession of Great Britain, the one place for the presentation of technical papers on all subjects, the place where wider discussion can be expected, and where greater merit, if deserved, will be awarded.

Does not this method of "sections" offer a common ground upon which all the societies can coöperate for the formation of not only a union building, but a united national engineering society?

Very respectfully, F. Stuart Williamson.

25 Broad St., New York, July 8, 1903.

(A union engineering building is one thing, and one great national engineering society, with four sections, is quite another. We fail to see why the union building can be of no value unless the several organizations are combined.—Ed.)

Some Comparisons Between the Original and the New Designs for the Manhattan Bridge.

Sir: Your issue of July 16 has a paragraph on the Manhattan Suspension Bridge over the East River, containing misinformation, which I beg leave to correct. The paragraph states that "full plans had been prepared for a wire cable suspension bridge" when I came into office. That was not the case. Tentative plans had been made and were under study, but none was in a condition for inviting bids. They contemplated a structure with a wooden floor and a working live load of 4,500 lbs. per lin. ft. of bridge (2 elevated tracks, 4 trolley tracks, 1 roadway, 2 sidewalks). The live load assumption did not seem adequate for the intended capacity of this bridge. Moreover, two more elevated tracks were needed to relieve more surely the old Brooklyn Suspension Bridge, only 1,500 ft. away. The structure must have ample capacity for all future demands in this location, and must be absolutely fireproof, safe and rigid in the case of accidental congestion of traffic, which the Brooklyn Bridge is not.

The new design (completed and ready to be bid on) provides therefore for a working live load of 8,000 lbs. per lin. ft. (4 elevated tracks, 4 trolley tracks, 1 roadway, 2 sidewalks), or 80% greater load than the original plans. It provides further, on the recommendation of the Board of Engineers appointed by Mayor Low, for an emergency congested load of 16,000 lbs. per lin. ft. of bridge. Under the emergency load, the deflection of the loaded middle span at highest temperature, with end spans unloaded (which is the worst case), will be only 23 ins. in a span of 1,470 ft., or, in other words, the suspension bridge will be as rigid as a truss bridge. The toe-pressure under the anchorage from that load will be only 11,800 lbs. per sq. ft. Were the old design, with stiffening trusses and larger side spans, to have been used, with sections and dimensions increased for the increased live loads, then the deflection of the middle span under the emergency load would have been over 90 ins., and the toe-pressure of the anchorage would have exceeded 17,000 lbs. per sq. ft. on sandy soil. A change in the length of spans and in the plans was therefore imperative.

You advise in an editorial article that bids be invited on plans with eye-bar chains and with steel wire cables. That sounds very plausible, but would lead to no benefit for the city.

In the first place, the design for the eye-bar chain, with positive pin connections for the web members, cannot be adapted for the straight wire cable. No connections have yet been invented for fastening the web members to the straight wire cables, and for positively transmitting the increments of the very large web strains to them. Friction devices or clamps for such large strains are not admissible.

Assumed, however, that such connections could be made in some other manner, a separate strain sheet would be required for the steel wire cable design, for the same form of stiffening. The same strain sheet cannot be used for chains and cables indiscriminately.

I have already indicated in the comparison of eye-bar chain and steel wire cable bridge (Eng. News June 25, 1903), that by reason of the greater extensibility of the wire and greater deflection of the cable bridge, the stiffening frame results heavier than in the chain bridge. That comparison, to be entirely fair, assumed the most favorable conditions for steel wire cables. If for the same design the unit strains in the cable were increased, then the deflection would increase and the sections of the stiffening frame must then be still larger and heavier to prevent overstraining of the metal in the bottom chord. If, on the other hand, smaller unit strains were used in the cable, then the cable sections would be larger and cost more, but the stiffening frame sections would not become light enough to make up for the greater cost of the cables. The only possible economy with steel wire cables is in combination with more flexible, that is lower, stiffening frames. But in that case the bridge will have much greater deflections than are desired for this structure, which is to be as rigid as it can be built for the colossal traffic expected over it. As a matter of fact, a cantilever structure had also been under investigation for its greater rigidity, before I assumed office.

It has been repeatedly pointed out before, that in the stiffening system of a suspension bridge, sections are determined by deflections, and deflections are determined again by sections. It is impossible to get away from that circle of conditions. For that reason, it is impossible to design a wire cable suspension bridge, which will, other things being equal, be as rigid as an eye-bar chain bridge. If, moreover, the superior chain bridge will be cheaper than the less rigid wire cable bridge, as I have shown in my comparison, published in your issue of June 25, what merit is there in this case remaining for the wire cable bridge?

It is a fact, that the suspension bridge, the oldest type for long spans and the easiest to erect, is the most intricate and difficult for accurate dimensioning. A suspension bridge can be built of any desired degree of flexibility or stiffness below that of an erect arch. In no other bridge form have so many mistakes been made by engineers as in the determination of the stiffening features of suspension bridges. The merit of the suspension bridge, when in competition with the truss, cantilever or arch type, depends entirely upon the stiffening system. The Manhattan Bridge will be as stiff as a truss or cantilever bridge, and yet cheaper than a wire cable bridge.

Mr. Hildenbrand, whose arguments in favor of wire cables were published in your journal, claims that the chain bridge will cost $3,000,000 more than an equally strong wire cable bridge. He had an opportunity, at his request, to explain his reasons to the above-mentioned Commission appointed by Mayor Low to pass upon the plans for the Manhattan Bridge. He failed to convince the Commission. It is not worth while to enter here into a long discussion of these reasons, but it is safe to say that he would fail to convince any unprejudiced and fair-minded experts. Later he addressed himself to the newspapers and to the Board of Aldermen, and asked them to decide intricate technical questions. I would advise him to present his arguments in a paper to the American Society of Civil Engineers. I believe that it would then be fully discussed, and the truth be more fully known, as it should be.

G. Lindenthal, Commissioner of Bridges.

New York, July 21, 1903.

Criticisms of the Report of the Special Committee of Engineers on the Plans for the Manhattan Bridge.

Sir: A commission composed of eminent engineers, none of whom had, however, any experience with suspension bridge construction, was appointed by Mayor Low to consider the feasibility of a design for the Manhattan Bridge, New York City, as submitted by Bridge Commissioner Lindenthal. The Commission was asked to answer the following questions:

(1) Are the plans in accordance with advanced knowledge of suspension bridge designing, with a view to economy of construction, provision for temperature stresses, rigidity under concentrated loads, and resistance to wind pressure; also as regards quality of steel and its protection against corrosion?

(2) Will the strength, stability and carrying capacity of the bridge be adequate for any congestion of traffic that may occur on the railroad tracks, roadways, and promenades?

(3) Will the structure, as designed, be fireproof?

(4) Do the plans permit of speedy erection of the superstructure, after the completion of the anchorages and tower foundations?

Two reports were rendered by the Commission, a preliminary report, published in Engineering News for March 12, and a final report, which appeared in your issue of July 2. Both reports fail to comply with the instructions as contained in questions 1 and 4.

It is a fact that "advanced knowledge of suspension bridge construction" does not lie in the direction of chain cables, but entirely in that of wire cables. Formerly chain cables were used, but such bridges have long since been removed and replaced by better designs. The Commission states that the use of eye-bar cables "is the main departure from the usual practice in suspension bridge construction." By this admission the Commission acknowledges that the design is not "in accord with advanced knowledge of suspension bridge designing."

It is a retrograde design!

The Commission has not reported on the economy of the design, as it has not made an estimate and compared it with a wire cable design. On the other hand, the Commission reported on self-evident points in the design regarding provisions for temperature and wind stresses and for rigidity under concentrated loads. The question of corrosion was entirely neglected. One of the members, now dead, had experience with corrosion of pin-connected bridges, but he was silent on this point.

The fact is that many pin-connected bridges have been removed because of excessive wear and corrosion of pins and of holes in eye-bars. The writer has seen pins grooved ⅝-in. deep by the eye-bars which they connect, while the bars were elongated in the eyes more than ⅛-in.

It is absolutely impossible to prevent corrosion between pins and eye-bars, and every engineer who has had experience in inspecting bridges, knows this to be a fact. How long will it be before the corrosion and wear necessitates repairs to the chain cables? We do know, however, that wire cables are entirely free from corrosion and can be thus maintained at a minimum cost.

There is also no question that a chain changes its shape under changes of temperature. Does this change take place in the pin joints or by bending of the bars? In the first case it will produce extraordinary wear between pins and eye-bars; in the second case it will produce secondary stresses not calculated on.

The Commission was satisfied with the tests of small sample bars, 6 ins. wide, and from the results obtained inferred that the strength of the 18-in. eye-bars would be the same.

This is erroneous. In the first place it is known to the writer that the testing machine in which the sample bars were tested loses by friction at least 15%, hence the strength of the sample bars is only 72,250 lbs. in place of 85,000 lbs., and in the second place, nobody knows what the strength of an 18-in. eye-bar will be. The yield point, given at 48,000 lbs. per sq. in., is also too high and would only be 42,000 lbs. if the friction of the machine were taken into account.

Steel containing 3½% of nickel in 8-in. bars, if properly treated, can be made to show a yield point of 80,000 lbs. per sq. in., and a tenacity of 110,000 lbs.

In regard to the time required for erecting the bridge, the Commission stated that "the design favors speedy erection of the superstructure after the masonry is ready."

This statement is, of course, merely a guess, as there is no precedent proving it, nor can the engineers of the Commission infer from their own experience. As a fact, the erection of a chain-cable bridge is the most indefinite matter, while that of wire-cable bridges is positively known, as demonstrated by numerous examples. The Buda Pesth bridge should have been finished two years ago, but the difficulties encountered in making the eye-bars and erecting the chain have delayed it till now. It is well known that a wire-cable design carried off the first prize in the competitive designs for said bridge, but eye-bars were substituted merely for favoring the home industries, as wire is not made in Hungary.

Another example of an eye-bar chain bridge, which was the cause of various troubles, is furnished by the Seventh St. bridge at Pittsburg, built by Mr. Gustav Lindenthal. The writer was in charge of the fabrication of the eye-bars for this bridge and knows that much time and extra labor were necessary to erect the chains. Moreover, the two chains are by no means working together as some bars are loose while others are overloaded. The double chain was intended for stiffening the bridge, subjecting alternately one or the other of the chains to compression. Not being built to withstand compressive stresses it happened that the chain buckled, causing much expense for repairs. In other words, the ingenious experiment failed. A still greater defect was found in this bridge, the anchorages being built with too small a margin of safety. At an unusual high water it happened that one of the anchorages commenced to slide forward and the whole bridge was in danger of collapsing if it had not timely been supported by false works and the anchorage been reinforced. In view of the figures, facts and past experience in the erection of suspension bridges it is almost incredible that the Commission of engineers has endorsed the chain cable design.

Gus C. Henning.

New York, July 18, 1903.

A LARGE ORE DOCK AT ESCANABA, MICH.; CHICAGO & NORTHWESTERN RAILWAY.

The iron ore traffic forms the most important part of the entire freight traffic on the Great Lakes, and special facilities are provided at several ports at the head of the lakes for loading the Michigan and Minnesota iron ores into vessels, while other special facilities for unloading these are provided at the lower ports, where the ore is used in neighboring plants or transferred to railway cars for transportation inland.

ORE SHIPPING DOCKS.

The loading facilities at the upper ports consist of large timber trestle piers with storage pockets or bins from which the ore can be discharged through chutes into the hold of a steamer or barge. The largest of these ore docks is the No. 2 dock of the Great Northern Ry., at Superior, Wis., which is 2,100 ft. long and has a storage capacity of 87,500 tons. Next to this comes the No. 2 dock of the Duluth, Missabe & Northern Ry., at Duluth, Minn.; this is 2,336 ft. long and has a storage capacity of 69,120 tons. The third largest is the No. 6 dock of the Chicago & Northwestern Ry., at Escanaba, Mich.; this is 1,920 ft. long, and has a storage capacity of 58,000 tons. This dock is the subject of the present article.

There are now 25 ore shipping docks, at eight

ports; they are owned by ten railway companies, and have a total storage capacity of 958,620 tons. A list of these docks, with their dimensions, etc., is given in the accompanying table, which has been revised up to May, 1903, and for which we are indebted to Mr. R. Angst, Chief Engineer of the Duluth & Iron Range R. R., of Duluth. Many of the docks have been rebuilt, including all those of the Chicago & Northwestern Ry., and the new dock described below takes the place of the old No. 2 dock. The Duluth & Iron Range R. R. has rebuilt docks Nos. 1, 2 and 4; No. 3 will be rebuilt next winter, and No. 5 probably in the winter of 1905-6. The docks of the Duluth, South Shore & Atlantic Ry. and those of the Wisconsin Central Ry. have been rebuilt. The others are the original docks. Most of the docks have a row of pockets on each side, but dock No. 1 of the Duluth & Iron Range R. R. has 1,076 ft. of double pockets and 312 ft. of single pockets. The angle of inclination of the pockets ranges from 38 degs. 40 mins. to 50 degs, 45 mins., but the majority have an angle of 39 degs. to 45 degs.

ORE DOCK NO. 6; CHICAGO & NORTHWESTERN RY.

The new dock No. 6, of Escanaba, Mich., which has been referred to above and is owned by the Chicago & Northwestern Ry., is a timber structure 1,920 ft. long, 50 ft. 2 ins. wide, 70 ft. high from the water to the deck, and 40 ft. from the water to the pin holes of the chute. It has 320 pockets, with a storage capacity of 58,000 tons. The approach is a double-track trestle 1,600 ft. long. About 150 ft. south of the dock is a rock-filled crib breakwater, parallel with the dock and 1,920 ft. long. At the end of the dock is a protecting crib.

Fig. 1 is a plan showing ore docks Nos. 5 and 6, with their track connections and their protecting breakwaters. Both docks are four-track structures. The plan also shows the new tie-preserving plant, with its tie yards and platforms, three retorts, and other equipment.

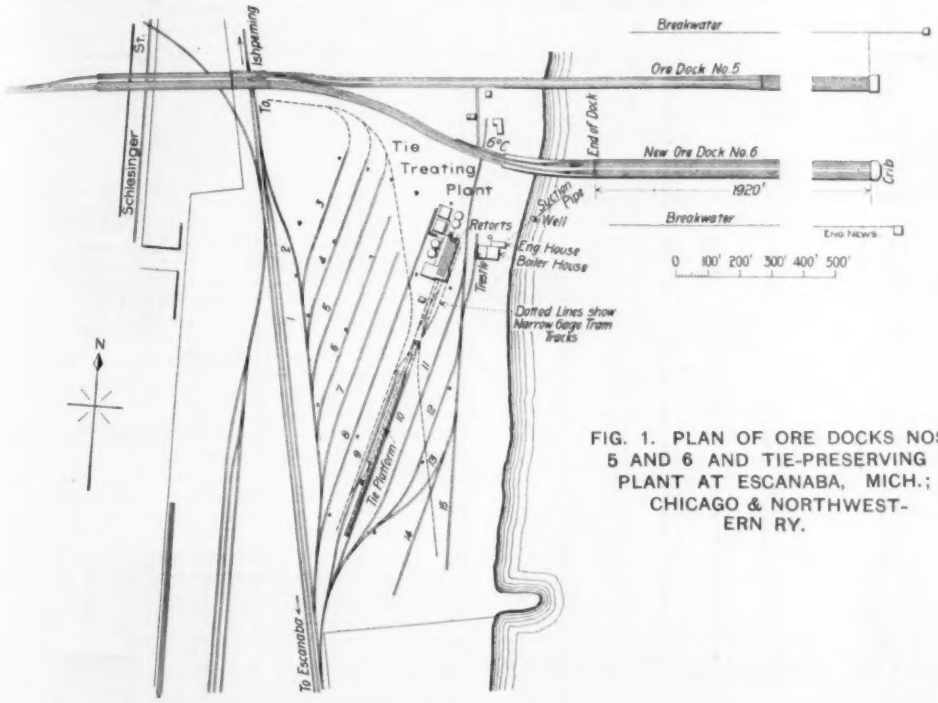


FIG. 1. PLAN OF ORE DOCKS NOS. 5 AND 6 AND TIE-PRESERVING PLANT AT ESCANABA, MICH.; CHICAGO & NORTHWESTERN RY.

The construction of dock No. 6 is shown very clearly in Fig. 2. Rows of piles 30 to 46 ft. long were driven in 20 ft. of water, the bents being 6 ft. apart; with 18 and 20 piles in alternate bents. Just above the water surface these piles are capped with timbers 12 × 14 ins., forming the sills for the framed bents of the dock. The posts and caps are largely of timbers 12 × 12 ins. and 8 × 12 ins., with longitudinal diagonals 8 × 12 ins., and transverse diagonals or sway braces 4 × 10 ins. The timbers are bolted and drift bolted together. The track stringers are laid on caps 12 × 14 ins., and are covered with 4-in. planks 5½ ins. long. Between the stringers are 12 × 12-in. fillers, over each cap, cut to a triangular section, so as to prevent the ore from collecting on the timbers. Two rows of 1¼-in. transverse rods, set up by turnbuckles, pass through the pockets and resist the bursting pressure of the ore upon the face of the dock. Outside of each track and between the tracks is a decking of 4 × 10-in. planks. Along each side is a row of fender piles, carrying three lines of waling timbers 12 × 12 ins.; and with horizontal fillers between the piles and vertical fillers between the wales. At the end of the dock is a rock-filled crib, having 46-ft. piles, and being sheathed with 4 × 12-in. pine plank above water and 4 × 12-in. hemlock plank below water. The outer corners of the crib are beveled, and protected by pile clusters.

The pockets have the sloping bottoms, at an angle of 45°, sheathed with 3-in. maple plank, laid lengthwise up and down the slope. The partitions and front of the pockets are of 3-in. pine plank. Vertically sliding doors cover the openings, through which the ore flows to the chutes. These doors are counterweighted. The chutes are also balanced, the chain from the end of the chute passing up to a pulley above the deck, down to a pulley below the pocket, up under the pocket to another pulley, and then down to a counterweight which hangs near the water line. The dock is equipped with the Denton chute hoist.

The material in the deck proper comprises 6,100 piles and about 9,000,000 ft. B. M. of timber; the approach has 300 piles and 1,300,000 ft. of timber; and the breakwater has 480 piles and 166,000 ft. B. M. This makes a total of 6,880 piles and 10,466,000 ft. B. M. Rock filling is placed around the heads of the dock piles, as shown. This filling required 26,491 cu. yds.; including 2,000 cu. yds. in the crib at the end of the dock. About 40,000 cu. yds. of material have been dredged to form a harbor, giving 20 ft. of water for a width of 100 ft. on the bottom on both the north and the south sides of the dock. This dredging was commenced July 7, 1902, and stopped for the season on Dec. 6, being recommenced in the spring and completed by May, 1903.

Work on the breakwater was begun on July 7, 1902, and completed Aug. 29. The erection of the dock approach, which is built in stories of about 16 ft. each, was done by putting the sub-bents together flat and hoisting them afterwards one story at a time. A force of ten men with one team packed and raised 50 of the second-story bents in eight days. The approach was completed about Jan. 1, 1903.

IRON ORE SHIPPING DOCKS ON THE GREAT LAKES.

Railway.	Location.	Dock.	No. of pockets.	Storage capacity, tons.	Length of dock, ft.	Height from water to Hinge of chute, ft.	Hinge of chute, ins.	Deck ft.	Deck ins.	Width over partition posts, ft.	Width over partition posts, ins.	Length of spouts, ft.	Length of spouts, ins.	Angle of pockets, degs.	Angle of pockets, mins.
Chicago & Northwestern	Escanaba, Mich.	No. 1.	184	24,104	1,104	28	10	48	6	37	0	21	0	39	30
" " "	" "	" 3.	220	30,284	1,356	31	2	52	8	37	0	27	0	45	0
" " "	" "	" 4.	250	32,750	1,500	36	6	59	2	37	0	30	0	45	0
" " "	" "	" 5.	232	43,152	1,392	28	6	53	3	37	0	21	8	40	0
" " "	" "	" 6.	320	58,000	1,920	40	0	70	0	50	2	30	0	45	0
" " "	Ashland, Wis.	" 1.	234	36,036	1,404	32	10	54	0	46	8	27	0	39	30
" " "	" "	" 2.	234	25,740	1,404	36	6	57	8	46	0	27	0	42	0
Total				250,066											
Duluth & Iron Range	Two Harbors, Minn.	No. 1.	202*	40,400	1,388	35	5	59	6	49	0	27	0	38	42
" " " "	" "	" 2.	208	41,600	1,280	33	5	57	6	49	0	27	0	38	42
" " " "	" "	" 3.	90	16,200	572	28	10	52	0	49	0	25	0	38	42
" " " "	" "	" 4.	168	36,960	1,112	37	0	62	0	49	0	29	0	38	42
" " " "	" "	" 5.	168	33,600	1,112	30	0	54	6	49	0	25	0	38	42
Total				168,760											
Duluth, Missabe & Northern	Duluth, Minn.	No. 1.	384	57,600	2,336	30	0	53	0	49	0	27	9	45	0
" " " "	" "	" 2.	384	69,120	2,336	32	0	57	6	49	0	27	9	45	0
" " " "	" "	" 3.	192	40,320	1,152	40	7	67	6½	59	0	27	9	45	0
Total				167,040											
Duluth, So. Shore & Atlantic	Marquette, Mich.	No. 1.	270	27,000	1,700	25	0	45	0	40	0	20	4	39	0
Duluth, So. Shore & Atlantic	Marquette, Mich.	" 4.	200	28,000	1,200	27	9	47	3	36	8	21	1	39	45
Total				55,000											
Lake Shore & Ishpeming	Marquette, Mich.	No. 1.	200	36,000	1,232	30	9	54	0	50	0	24	7	38	40
Great Northern	Superior, Wis.	No. 1.	250	40,500	1,525	32	0	57	0	49	8	27	2	45	0
" "	" "	" 2.	350	87,500	2,100	40	0	73	0	62	8	32	4	45	0
" "	" "	" 3.	160	40,000	960	40	0	73	0	62	8	32	4	45	0
Total				168,000											
Minn., St. Paul & S. S. Marie	Gladstone, Mich.	No. 1.	120	15,000	768	26	8	47	0	37	0	21	8	40	0
Wisconsin Central	Ashland, Wis.	" 1.	314	48,356	1,908	40	0	66	2	36	0	27	0	50	45
Chicago, Mil. & St. Paul	Escanaba, Mich.	" 1.	240	50,400	1,500	40	2¾	66	6	52	0	27 & 29†		45	0
Algoma Cen. & Hudson Bay	Michipicoten, Ont.	" 1.	12	.....	312	34	0	43	4	25	0	22	6	44	0

*No. 1 dock, D. & I. R. Ry. has double pockets for 1,076 ft., and single pockets for 312 ft.
†The dock of the C., M. & St. P. Ry. has 120 spouts 27 ft., and 120 spouts 29 ft. long.

Pile driving for the dock was begun on Sept. 27, 1902, and finished on Dec. 17. All the dock piles were driven by the use of a pump and water jet, with the exception of a few at the west end of the approach. The superstructure of the dock proper was started Dec. 29, 1902, and finished May 1, 1903. The timber was framed ashore and hauled by teams on the ice along the dock, and thence hoisted by steam derricks, one piece at a time, to its position in the work. Two derricks were used, one to raise the first-story bents, while the other followed raising the second. The first derrick was then started on the third story, and the second followed with the fourth or last story. Decking was then put on, track laid, and plank for the pockets (cut to shape by a saw mill in the framing yard) was taken on flat cars up on top of the dock, where it was delivered and placed in position. Spiking and bolting crews followed the work of erection. Each derrick put in place about 80,000 ft. of timber per day with 12 men and four teams. The working force at its maximum was 175 men, reduced to 125 when the timber frame was all erected. The plank was all completed April 15, and the ironwork of the pockets was completed in May.

The dock was designed under the direction of Mr. E. C. Carter, M. Am. Soc. C. E., Chief Engineer of the Chicago & Northwestern Ry.; and Mr. C. Palmer is the Division Engineer at Escanaba. The contract for dredging was given to the C. H. Starke Dredge & Dock Co., of Milwaukee, Wis. The contract for the construction of the dock, approach and breakwater was given to M. J. Peppard & Co., of Minneapolis, Minn.

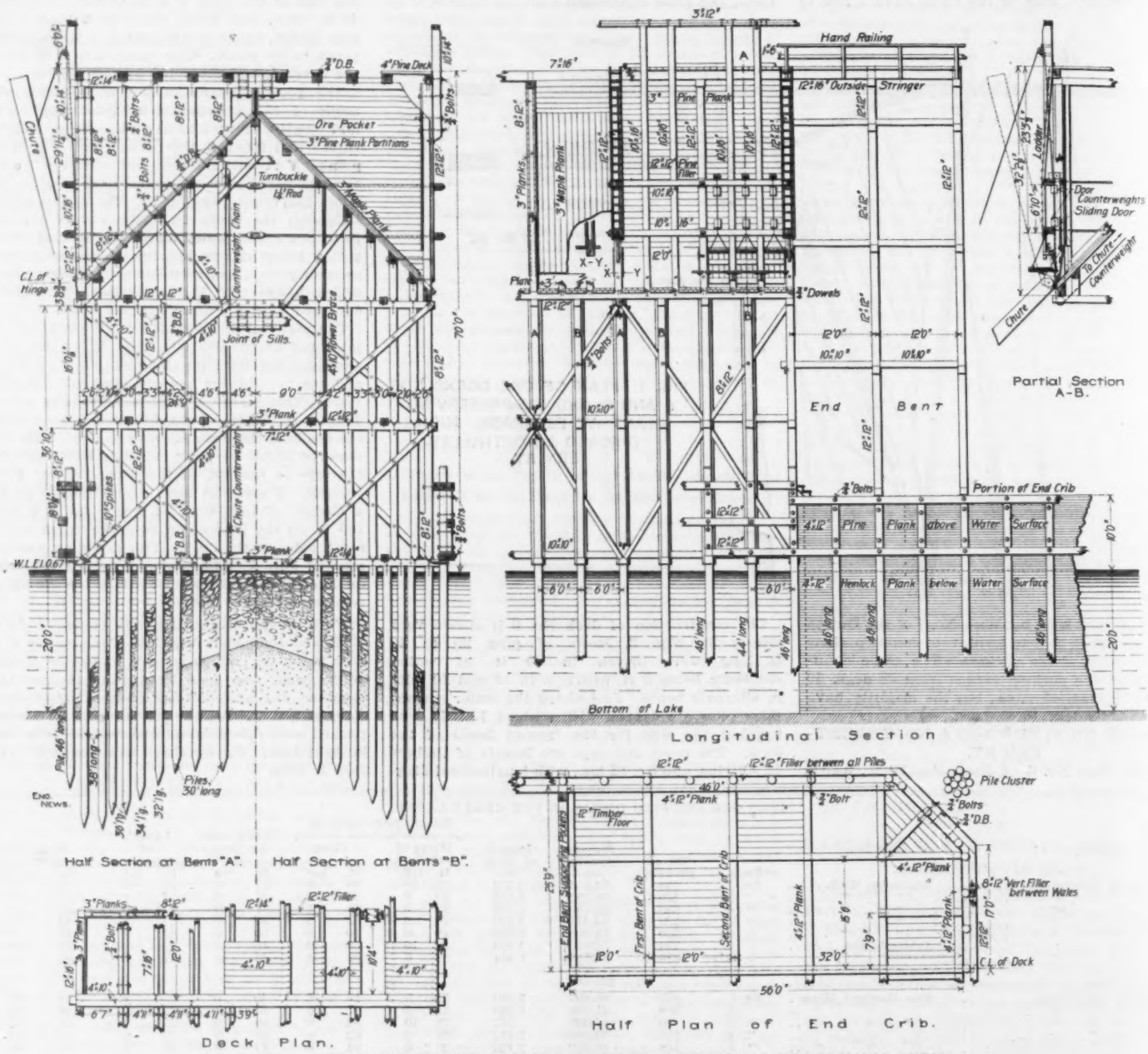


FIG. 2. NEW ORE DOCK OF THE CHICAGO & NORTHWESTERN RY. AT ESCANABA, MICH.

E. C. Carter, M. Am. Soc. C. E., Chief Engineer, Chicago & Northwestern Ry. M. J. Peppard & Co., Contractors, Minneapolis, Minn.

SPRINGS AND SPRING STEEL.*

By Wm. Metcalf, M. Am. Soc. C. E.†

Thirty years ago, when the writer first became interested in springs, Bessemer and open-hearth steel were undeveloped infants and their uses were mostly experimental for everything except rails and plates. Bolster springs were made mostly of gum, gum and coils of steel around them, and gum in boxes, in great variety; also of coiled springs of astonishing variety. There were helices and volutes, round, square, flat, beveled, oval, egg-shaped and nondescript sections of bars, used for both bolster and draw-bar springs, made of either the old style German steel or of crucible steel. There were double, single and half volutes and helices in groups or single coils, and others of different diameters one within the other. For elliptic springs we had flat, concave, ribbed and corrugated bars. Theee springs were covered by patents as numerous as the varieties; and, of course, each kind of spring was the best in the market.

Specifications for springs were limited to space to be occupied and load to be carried; it was go-as-you-please for the manufacturer, scrambling and fun for the traveling men, and all around confusion worse confounded.

The gradual introduction of open-hearth and Bessemer steels, sure winners because of their cheapness, promptly drove out the gum altogether, and more slowly but inevitably drove out the crucible steel. Crucible car-spring steel is still heard of occasionally from the few who have not advanced with the art. The first effect of the introduction of these cheaper steels was breakages innumerable, until careful railroad men were driven nearly wild.

Before crucible steel was entirely driven out one maker was asked to make the lightest possible spring to go into a given space and carry a given load for the then common 30,000-lb. freight car. The result was 32 lbs. to a group, 128 lbs. to a car; they worked admirably, the springs were adopted and many thousands were ordered from different makers. A few months later the same maker was asked to make the heaviest spring he could put into the same space for the same load, because the light springs were breaking faster than they could be replaced. The result was a nest that weighed 72 lbs., or 288 lbs. to a car, a gain of 160 lbs. for the manufacturer. Shortly after he was sent for again to consult about a proper specification, to which he replied that the proper specification was to specify his springs, and he was told promptly that the great railroads of the country could not, and would not, be tied to one concern; that although they were buying springs on a five-year guarantee, the breakages were so great, even of the heavy springs, that to insist upon the guarantee would, they were sure, break up every one of the spring makers, and that would be the worst break of all. To take a back step and return to

*A paper read before the American Society for Testing Materials, at Delaware Water Gap, July, 1903.

†Pittsburg, Pa.

gun and crucible steel exclusively would be a confession of weakness in the mechanical department amounting to almost imbecility, and there seemed to be no relief except through a scientific study, careful experiment, and proper specifications.

This was the beginning of specifications for springs and probably the leader for many others. The first specification of which the writer has any knowledge was for the steel, and was due to our worthy President. He wisely ignored the mode of manufacture and devoted himself to getting the best material consistent with reasonable cost of manufacture, not demanding something impracticable, not expecting a dress suit at a shoddy price, and equally determined not to have shoddy when he paid for good wool. For one, the writer kicked against the close limit on carbon, and all to no purpose. The result was the now famous and almost universally accepted Pennsylvania R. R. specification for spring steel, now known everywhere as the "Standard Specification Spring Steel," and it has proved to be entirely reasonable because it can be made for a reasonable price, and when it is well made it is entirely satisfactory.

Following this, there came from the mechanical department of the same railroad the first reasonable and sensible specification for coiled springs. A circular section, the round bar, was adopted for all coils because the strain was torsional and the round section gave the maximum resistance to torsion. Calculated from the best known formula for torsion a complete set of springs was designed for the various uses about a car. Upon the first trial the springs proved to be about 33% too strong; this led to a discussion between the spring makers and the railroad engineers as to who was at fault, and, naturally, each fellow insisted that he was right and the other fellow was wrong. The writer took the matter up by making some springs that were of the specified composition and the bars of the specified size as exactly as they could be rolled on a large scale, not going into a refinement of thousandths of an inch; then he saw that they were treated properly, both in the coiling and in the hardening and tempering. They were certainly about 33% too strong.

This led to a consideration of the formula; it seemed at first sight that if torsion were the only resistance the closing down of a coil must lead to a compression of the tempered steel that was simply impossible. Then where did the steel go in compressing the spring? Probably the ends slipped around the base, adding so much to the coils. A spring was placed in the testing machine between two clean, smooth steel plates, and the position of the ends marked. Upon compressing the spring the ends did not move. Next the pressure was released and by means of a small square set against the sides of the spring the diameter was marked on the base plate as well as the ends; then the spring was closed again and again the ends did not move. A trial with the square now showed that the spring had increased in diameter at its middle height ¼-in.; it was barrel-shaped. The spring was 6 ins. in diameter and 8 ins. high; this meant that the pressure had bulged a 6-in. half-circle arch ⅛-in. at the crown, or it had apparently expanded a 6-in. circle ¼-in. in its diameter. This accounted for the increased strength above the formula requirement, and also when the writer considered the complication of strains involved in a combination of torsion and flexure produced by an end pressure, it drove from his mind any further consideration of a formula. There may be mathematicians who could figure it out, but the writer is certainly not one of them.

Assuming now that the springs are formed either by coiling the helices or arching the plates (of elliptics), the next operation is the hardening and tempering. Although the differences in temperature necessary to produce the best hardening for different quantities of carbon are apparently slight as shown by the heat color, they are very important, and are best acquired by the experienced eye possessed by a man of good judgment and not color blind.

COILED SPRINGS.

The hardening and tempering of coiled springs is a comparatively simple matter if the temperer knows the carbon with which he is dealing.

The objection to the carbon limits mentioned before was that we had been accustomed to making springs of every carbon from 0.60 to 1.30 and the limiting of carbon to 0.90 to 1.10 threw out of use a large amount of steel. Now that specifications have been changed so that coiled springs may be of any carbon from 0.70 to the highest, that objection has been removed, and the results will be just as good if the best conditions are observed. The lower carbons should be put into the larger bars, because the largest bars are the most difficult to harden safely, and the difficulty increases in a geometrical ratio with the increase in carbon. A good rule is to put the 0.70 to 0.90 carbon into bars of more than 1 in. diameter; bars from 1 in. to ¾-in., 0.90 to 1.10 carbon; bars from ¾-in. to ½-in., 1.10 to 1.20 or even 1.30 carbon; and little rods below ½-in. of any high carbon up to as much as 1.45.

This was the old practice with crucible steel, and the spring makers were always informed of the carbon they had to deal with; the result was that breakages in the shop or failures in service were very rare. Steel of 0.60 to 0.90 carbon was hardened in water, sometimes with about 0.90 carbon a film of oil was used on the water. From 0.90 to 1.10 carbon about four or five inches of oil was used on the water, and for higher steel oil was used and kept cool by an external tank of cold circulating water, or by a coil of pipe inside of the tank with cold water running through.

Coiled springs require so little manipulation after coiling that they are usually at a proper heat for hardening as they leave the machine; care should be taken, however, in heating for coiling, that they are of an even temperatur throughout the bar, and neither too hot or too cold. If they are too hot the grain will be coarse and fiery, and even if they do not happen to break in testing they will be brittle and liable to break in service. If too cold they will not harden thoroughly, and if they do not set in testing they will probably set in service.

As to the heat, it needs to be slightly higher for mild steel than for high carbon; in general it should be just a slight shade above the recalescent point, but that is a matter not readily grasped by the average worker, and a good practice is to aim for a nice, medium orange color for high steel, and a little brighter orange for the milder steels. Good sense and experience and honest attention to the work are the best guides.

Tempering must be suited to the carbons; 0.70 to 0.80 carbon will require very little drawing, 0.90 to 1.10 may require the oil on them to flash, and for higher carbons the oil may be burned off: above 1.30 carbon a heat that barely begins to show color will generally give a good spring temper. In tempering, as in hardening, good sense and good judgment are the best guides.

Helical extension springs require more care and exact treatment than compression springs; whether it be that they are liable to be pulled out so as to be strained beyond the elastic limit, or that the uncoiling is in a continued direction of some strains set up in coiling and so causing more injurious strains, the writer cannot say, but a large experience has demonstrated clearly that extension springs as compared to compression springs are much the more troublesome, so that great care must be used to produce satisfactory results.

ELLIPTIC SPRINGS.

In dealing with elliptic springs the problem is somewhat different from that of coiled springs. There are many more pieces to deal with and each one must be shaped, hardened and tempered separately and then all assembled and brought to a neat fit. The heating for shaping, and hardening and tempering should be done carefully, and in general the same as has been described for coiled springs.

It seems to be the fact that there is comparatively little trouble with double elliptics, because, probably, when they are closed down solid they have a bearing upon their two parts, just as a coiled compression spring has, and therefore they are not liable to be strained beyond the elastic limit unless they receive a blow that is powerful enough to crush and break the material. The case of the semi-elliptic, where the spring rests on the middle with the ends turned up to receive the load, as in a locomotive, is very different and is comparable to the case of the coiled extension spring; there is no final bearing for the ends, and by a sudden jar causing an excessive motion they may be strained away beyond the elastic limit and broken. It is understood that these springs now give more trouble than any others and that to get them satisfactory is a serious problem.

The standard specification for locomotive spring steel retains the original low limits for phosphorus, sulphur, silicon and manganese, and carbon between 0.90 and 1.10; this is well, first to insure good material as far as it is possible to get it, and, second, because all of the bars are flat, of nearly the same section, and there is no reason for allowing a wider range in carbon; steel can be had within these limits without any increase of cost, and now that coiled springs may be of any carbon from 0.70 to the highest, all off heats can be used for helicals without any loss to the steel maker.

It is a common practice to form the plates in a mold, take them out and adjust them to a templet; twist them a little here, pound them with a hammer there, hold another spot with cold tongs, and, finally, when the piece is shaped to the taste of the operator and is of many different colors of heat, and strained here and there by hammer blows, to plunge them into the oil bath to harden them.

This is certainly all wrong; it is a quick, cheap way to get out a big product and it is sure that every plate so treated is dosed with injurious strains that may result in a mysterious and unaccountable fracture. Plates so manipulated should be put into a proper furnace and brought to a uniform, correct heat before quenching; this heating would remove the uneven strains and make a much better plate. This would require a little care, a little time and possibly a small increase in cost; the question is, would it pay if it increased the life of the spring, reduced the number of breaks and kept the engines longer on the road?

Another point needs attention when hardening in oil both for helices and elliptics; the oil should be watched, a little fresh oil should be added every day, and, finally, when the whole mass has become pretty well burned, so that it appears as if it were mixed with sand, it should be thrown out, the tank cleaned and filled with fresh oil. Worn out oil loses its power of convection largely, and will not harden the pieces as they should be; if it is attempted to correct this by having the springs a little hotter, then the grain will be raised and the result will be fiery, brittle plates, and more mysterious breaks.

Compared to a fine tool, a razor, or any similar article, a locomotive spring is a comparatively coarse article, but nature did not make one set of laws for steel for milling cutters, taps, reamers, etc., and another set for steel for locomotive springs; the laws are all the same, the strains are the same, the sensitiveness is the same. The average spring steel contains more impurities, phosphorus, sulphur, and so on, than fine tool steel, and just for that reason it is not only not so strong in the tempered condition as the finer steel, but it is also far more sensitive to uneven strains and will break under conditions that a piece of finer steel would endure safely.

In the absence of laboratory tests and exact data on these points it may be well to illustrate them by the action of clock springs and watch springs which are fine enough to answer the purpose of laboratory work.

A LESSON IN WATCH SPRINGS.

These springs are spirals and when well made are really wonderful in their work; probably every one here has a watch which runs year in and year out with great regularity, and the running is done by a little spring that drives machinery of many times its own weight often with a variation in speed of only a few seconds a month. A watch main spring about .002 or .003 of an inch thick, less than ⅛-in. wide, and several feet long should be made of steel of 1.30 to 1.49 carbon.

The first attempts to make this steel in the United States led to trouble of course; the steel was not uniform in quality and was not properly melted because it showed all sorts of temper in the same spring. It developed some other tempers also.

The requirement of the spring maker was that the spring must be a perfect spiral; when pulled out straight and allowed to snap back it must coil into a spiral of even space between the coils through its whole length; also it must, when tempered, be of a perfectly even beautiful blue color. Such springs were submitted to the steel makers and they certainly were admirable works of art.

The American springs were any shape as far as even coiling went; they touched at one spot, were too far apart at another and so on, were any color from a dark, dingy blue to a brown; therefore, the steel was not of uniform quality and so badly melted that the carbon was not evenly distributed. There was no room for argument, the springs told their own story. The steel makers knew that the steel was uniform in quality and in carbon, that it was thoroughly well melted and that the carbon was evenly distributed as it was possible to have it in any steel. It was now their business to find the cause of the trouble.

After two or three years of struggling with the problem the cause was found in the annealing. The steel was cold rolled in long strips about three inches wide and one hundred to two hundred feet long. To get it down to .003 of an inch thick it had to be annealed, pickled, washed in lime water and baked about six or eight times, and it can be readily seen that this required great care, skill and close attention. The annealing was done in boxes, packed with coils of steel, filled in with fine charcoal, and the cover was luted on carefully. In spite of all care the troubles recurred, some coils made beautiful springs and others were worthless; finally a wise and expert drawer of fine wire, Mr. Edwin Kidd, now of the Globe Wire Co., asserted positively that steel never could be made right by annealing in closed boxes, it must be annealed in an open furnace so arranged that no flame could strike the steel directly, and yet not in a muffle. Then the operator could watch each coil turn it and watch it until it was heated just right and then remove it from the furnace and place it in a warm, dry place to cool slowly. It was thought that his plan was impracticable, but the case was desperate and the plan was tried with most remarkable results; it was a brilliant success, and that steel soon became famous for its excellence.

What is the explanation? It is very simple when you know, but it was a hard road to travel to the knowledge. In annealing in boxes the steel could not be seen and the heat could not be known; then the charcoal would become incandescent and run the heat too high, and the coil would be much hotter on one side than on the other: that meant that part of the coil would be of one grain and part of another grain and greatly different structure. This difference in structure would remain in the steel until the spring was finished, ready for hardening and tempering; the heating would require but a few moments and when heated the spring would have to be quenched immediately because its small size admitted of no time for manipulation. Now, although the structure of steel changes rapidly in answer to any change of temperature, yet it does require a little time, and in this case that little time could not be allowed, the grain of the spring could not be evened up and the result was an uneven spiral, varying color, and a bad spring all round, until the steel was annealed to an even grain and structure.

Consider this now as a long-continued, worrying and costly laboratory experiment; and apply the facts to your car and locomotive springs; you are dealing with the same material, it has the same properties, the same forces are at work, and the same results will be had in greater or less degree. Possibly if you consider it seriously you will conclude that it will be worth while for you to be careful in the manipulation of the springs which you wish to have to carry your engines and cars in safety.

TESTING.

Testing of springs is probably well understood and need not be enlarged upon to any great length.

All springs are made higher than the finished height to allow for the initial set which occurs when they are first closed down; this is necessary because if the temper were left so high that they would not set they would nearly all break either in the test or in a very short time in service. It is important in testing that springs be held down solid for a few minutes to allow for lag; a spring may be in a state of unstable equilibrium and endure a quick closing and release without setting too low if soft or breaking if hard, and then soon fail in service; this condition can be detected by holding it down solid for a few minutes, giving the necessary time for the strains to develop.

The United States Government required springs for mortar carriages to be held down solid for sixty hours. This is just as unreasonable as not to hold them down at all; it is not on record that any spring broke after the first five minutes, but the holding them down for a few minutes under a heavy pressure is important.

CHEMISTRY.

While it is not well as a general rule to specify a given chemical composition and a physical test, unless the engineer is a very expert steel maker, it certainly was wise to fix a reasonable maximum of allowable phosphorus, sulphur, silicon and manganese, and a reasonable range of carbon; the effect of these elements is well known and excess can be guarded against, as it has been in the standards now adopted by all careful railroads, without any excessive cost or trouble to the manufacturers. It is assumed, of course, and properly, that if the chemistry is correct, the physical condition good, and the sizes accurate, the steel maker's responsibility ceases, and as a general thing the rule works well, but it has its exceptions. Not long ago a prominent spring maker received several carloads of steel which was well within the chemical limits, was sound, and rolled accurately; upon attempting to work it into springs the blanks nearly all broke in the process of forming. The steel was excessively red-short and it was returned to the maker. The cause of this was back of the engineer and the chemist, at a point probably impossible to reach by specifications, except to claim the right to test for red-shortness, and this should be done.

There can be little doubt that this trouble was due to excess of oxygen, an element that cannot well be determined by ordinary analysis. That an excess of oxygen will produce excessive red-shortness is beyond dispute. After many years of analyzing and experimenting by Prof. John W. Langley, supplemented by many tests in the shop, to locate if possible the cause of the difference in strength in the tempered condition between Bessemer, open-hearth and crucible steels of practically even composition, the conclusion was reached that the cause was to be found in the difference in the quantities of oxygen, nitrogen and hydrogen found in the steels; in Bessemer steel great quantities of these elements are blown through the mass; in open-hearth great quantities flow over the surface of the steel for hours and much is absorbed; in the crucible only the amount that is in the pot or that may pass through the sides can get into the steel. That these elements make a great difference is easily observed when a crucible happens to become uncovered during the melting, as sometimes occurs; if a hundred ingots be topped and set up for inspection and only one has been exposed to flame by the crucible becoming uncovered, the inspector will notice it immediately, mark it "gas" and relegate it to the scrap heap. There is no good reason apparent why an atom of oxygen, nitrogen or hydrogen may not be as potent as an atom of phosphorus, sulphur or silicon, and they are all present in greater or less quantity. Prof. Langley's conclusion was that oxygen is the head devil; the writer held to nitrogen for a long time as his pet mischief-maker, but it seems probable that Langley is more nearly correct.

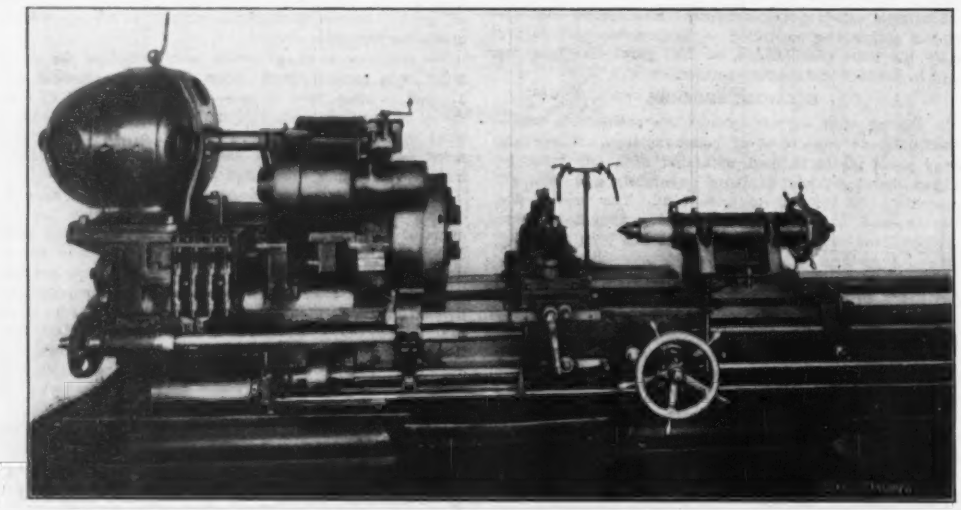
These remarks are not meant to belittle Bessemer or open-hearth steel in any way; their great merits and usefulness are too well established to leave them open to criticism. The trouble is not in the method, it is in the man. Given the best material that the world can produce and it will not make good crucible steel if it is not melted properly, and no subsequent treatment will make a badly melted ingot a piece of good steel. It is the same with Bessemer and open-hearth; bad blowing or bad melting cannot be cured by good chemistry. The red short steel referred to doubtless came from a wild heat; the melter knew it, the roller knew it, and probably the superintendent knew it; they were all working for product and they knew how to work that steel into pretty bars, but the spring maker did not know how to work it into springs and it is probably well for his employers that he did not know.

We have now covered the ground that belongs properly to the engineer, the spring maker and the inspector, and have tried to show how the work should be done, how to detect errors and how to correct them. This brings us back to the steel maker and out of the domain of the steel users; all steel is made good or bad in the crucible, the open-hearth furnace or the Bessemer converter; if the steel is good there can be no excuse for bad springs, and if the steel is not made good by good melting or blowing it cannot be made good by any subsequent treatment.

This matter rests then with the steel maker, because it will not do for the engineers or inspectors to attempt to regulate shop practice unless they are ready to assume all of the consequences. During the latter part of the Civil War the government sent construction officers to the gun foundries; these gentlemen were to see that the right kind of iron was used, and to see the guns cast. They did see that iron of the recognized standard brands was put into the furnaces, and it is needless to say that they did not know anything about its quality. They also saw the guns cast; this occurred after noon, and sometimes when the furnaces were slow they got in a hurry to go away and would say it was time to cast the guns; the answer always was: "All right, give us a written order to cast the gun, you agreeing to accept the result and we will cast at any time you wish." The reply followed: "No, you must submit the gun to all regular tests, of course." And the argument ended: "Very well, if we are responsible we will use our own judgment until the gun is submitted for inspection." The same rule must apply to any manufacturer; if he is to be held responsible he must not be interfered with in his management.

A good melter, of sound judgment and correct eye, knows exactly what he is doing and precisely what he will get; there is no pyrometer or spectroscope that is equal to a well-trained eye to inform one what is going on; and it is doubtful if any machine will ever be invented that will be as useful as a good eye. All that the steel user can do is to plead for good work, and be willing to pay a decent price for it; the rush and drive in the mills, the keen, fierce competition of different manufacturers and lively salesmen, and the desire of the purchasing agent to keep down costs, are potent factors for poor work. The engineer wants the best he can get for his purpose and he often finds that these factors have placed him between the devil and the deep sea.

It is hoped that what has been said will lead properly to the conclusion that nothing has been suggested that will cost anybody one cent; it is meant only as a plea for honest effort, reasonable care and close attention to every detail. These will result in general satisfaction, and any other course will lead to disaster.



26-IN. LATHE FOR HEAVY SERVICE, WITH DIRECT-CONNECTED MOTOR AND MULTIPLE-VOLTAGE SPEED CONTROL.

A HEAVY-SERVICE LATHE WITH ELECTRIC DRIVE.

The view shown herewith illustrates well the new style in machine-tool building due to the advances made in the past few years in shop methods. Air-hardening and other special tool steels which permit of very high rates of metal-removing, that is heavy cuts and feeds and great cutting speeds, require heavier lathes and planers than have been standard. The old machines are not sufficiently rigid or massive in frame or strong enough in drive and gearing, to withstand such cuts and feeds; in consequence new machines, stronger and heavier throughout, were called for, and many builders of machine tools have met this demand by bringing out a special line of machines.

Direct driving by electric motors has also established itself as a requisite in modern shops. It decreases the cost of the power required by eliminating the greater part of the transmission losses (in shafting, etc.), and adds to the convenience of handling the tool. A large part of the belting is done away with, and crane service becomes available over the whole shop area. Both direct and alternating current have found successful application to machine-tool driving, though the former is considerably in the lead. With either system large tools are advantageously driven by a motor direct-connected to the driving shaft of the machine; smaller machines are often grouped, with a common drive-shaft taking power from a motor. In arranging large machines for electric driving, the conversion has often been accomplished by setting the motor in the base or on a bracket attached to the machine, which otherwise was identical with a belt-driven machine. In other cases, however, the machines were specially designed for electric motor drive, without reference to the forms developed for belt-driven machines.

The machine shown herewith is of this class. The usual cone-pulleys in the headstock are replaced by a seat for the driving motor and its gearing, and the electrical control switches. The machine is designed for the heaviest service which two tool-posts can require. The lathe, of 26-in. swing, was made by the Bullard Machine-Tool Co., of Bridgeport, Conn., while the driving motor and controller were supplied by the Crocker-Wheeler Co., of Ampere, N. J. The mechanical features of the machine comprise (besides the general strong and heavy construction for most rapid production) two tool-posts, front and rear, which may be fed simultaneously inward or outward, a two-speed gear change in the transmission from motor-shaft to spindle, a hand traveling attachment for the tail-stock, and a lubricator pump belted from the motor-shaft. The driving motor is a shunt-wound machine rated at 20 HP. Rigidly connected with the armature shaft is a pinion-shaft carrying two pinions; by shifting these in one direction or the other they engage with corresponding gears on an intermediate shaft, which drives the spindle by a spur-pinion. The motor is supplied by a four-wire system, which makes six different voltages available. Since each voltage has a definite motor speed corresponding to it, there are six speeds possible without employing resistances; a field rheostat for weakening the field circuit serves to give the intermediate speeds. The two-speed pinion gearing again doubles this range of speeds. The changing of connections is all done automatically by a controller mounted at the front of the headstock below the leadscrew. As may be seen in our photograph, there is a third longitudinal rod in front of the machine, in addition to the leadscrew and the feed-shaft. This rod gears to the spindle of the controller; it is operated by

bevel-gears from a vertical hand-lever at the right of the apron. In this way the lathe operator has the speed control right at hand, without moving away from his work.

RATIONAL FORMULAS FOR THE STRENGTH OF A CONCRETE-STEEL BEAM.

By S. E. Slocum,* B. E., Ph. D.

The subject of concrete-steel construction has recently developed with great rapidity, but is as yet upon a very unsatisfactory basis as regards structural dimensioning. Formulas derived by rigorous mathematical development from purely theoretical assumptions are too complicated to be of much practical value to the engineer. On the other hand, empirical formulas are not valid outside the region for which they were determined, and no matter how extensive the series of experiments upon which they are based, they will always be unsatisfactory by reason of their inflexibility. The most scientific and satisfactory basis for any branch of engineering design is a method of simple, rational analysis, founded upon elementary assumptions and involving a few empirical constants which can be readily and accurately determined by experiment. The object of the present paper is to develop such a method for the analysis of a simple form of concrete-steel beam.

The first assumption that will be made concerning the behavior of the material under stress is Bernoulli's assumption that cross-sections of a beam which are plane before flexure remain plane after flexure. The assumption of Hooke's law of proportionality of stress to strain was formerly the starting point for all investigations in the strength of materials. For cast-iron, stone and cement, however, it has been found that Hooke's law does not hold true; but as yet no entirely satisfactory law has been found to take its place, although several have been proposed; e. g., the parabolic, hyperbolic, cubic-parabolic, exponential laws, etc.* The curve of normal stress distribution over any cross-section of the beam has, in fact, no simple equation. All that can be said of it is that the more constants contained in the equation assumed to represent this curve, the more nearly can it be fitted to the actual curve. Thus all proposed laws have the character of interpolation formulas, and are only approximately true within narrow limits. In view of the complicated and inaccurate nature of these laws, Föppl, one of the foremost German authorities on the subject, strongly recommends that general investigations in the strength of materials be carried out only upon the assumption of Hooke's straight line law, and that the results so obtained be then modified by a factor corresponding to the factor of safety, obtained by a comparison of theoretical results with those of careful experiment.† In this way the determination of an empirical factor of safety is substituted for the determination of an empirical law of distribution of stress. Thus simplicity of calculation is secured without the sacrifice of rationality of the formulas.

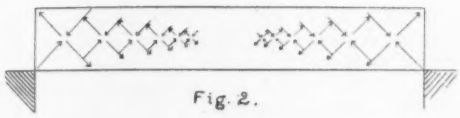


Fig. 2.

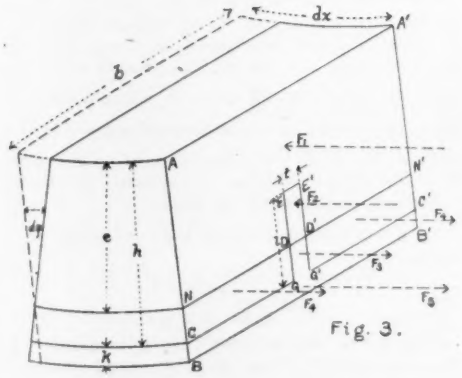


Fig. 3.

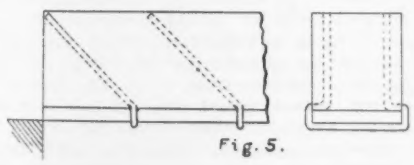


Fig. 5.

In the present discussion, therefore, the second fundamental assumption will be that of Hooke's law of proportionality of stress to strain. The effect of this assumption will be to make the moment of resistance of the beam slightly less than if the parabolic, or one of the other laws, was assumed, thus causing an error on the side of safety by requiring a larger beam for a given loading than if some other law was assumed. The nature and amount of the error involved in the above assumption is well shown by the experiments of Melan, who found the stress in the extreme fiber of a concrete beam to be .97 of its value as obtained from the ordinary bending formula for beams.‡

Consider a beam of rectangular cross-section, resting on vertical supports and carrying a vertical load, and assume a system of rectangular coordinates; the X-axis being horizontal and lengthwise of the beam, the Y-axis vertical, and the Z-axis horizontal and perpendicular to the plane of X Y. If a section of the beam is made by a plane parallel to the plane of Y Z, the internal stresses acting on either face of this section can be reduced in general, to a single force and a couple, which hold in equilibrium the external forces acting on the same side of the section. The single force acts in the plane of the section, and is called the shear, while the forces of the stress-couple are normal to the section, and their moment about the neutral axis of the section is called the moment of resistance of the section.

It is proved in mechanics that if q is the intensity of the shearing stress on an infinitesimal area d F at the distance u from the neutral axis in the cross-section of a rectangular beam of breadth b, then

$$(1) \qquad q = \frac{S}{bI}\int_u^e y\,dF = \frac{S}{2I}\left(e^2 - u^2\right)$$

where S is the total shear on the section, I is the moment of inertia of the section about the neutral axis, and e is the distance of the extreme fiber, from the neutral axis.‡ It is also proved in mechanics that if a shearing stress exists at any point in a plane section of an elastic solid, there is an equal shearing stress acting at the same point in a plane at right angles to the first.§ Consequently, if three plane sections of the beam are taken, parallel to the three coördinate planes, respectively, the stresses acting on these planes will be as follows: On a plane section parallel to the plane of Y Z; that is to say, on a vertical cross-section of the beam, there are normal stresses forming a stress-couple about the neutral axis of the section, and shearing stresses paral'el to the axis of Y and varying in intensity with the depth as the ordinates to a parabola having its vertex in the neutral axis; on a plane section parallel to the plane of X Z; that is to say, on a horizontal longitudinal section of the beam, there are shearing stresses parallel to the X-axis and equal in intensity at any point to the intensity of the vertical shearing stress acting at the same point; while on any plane parallel to the plane of X Y; that is to say, on a vertical longitudinal section of the beam, no stresses act.

These shearing stresses acting in planes at right angles to one another give rise to tensile and compressive stresses. For consider a small triangular prism of which Fig. 1 is a projection on the X Y-plane. Let q be the intensity of the vertical and horizontal shearing stresses acting at the point O, φ the angle which the inclined face makes with the horizontal, and p and q' the intensities of the normal and shearing stresses, respectively, acting on this face. Now p and q' can have no components in the direction of the Z-axis, for there are no forces which could hold them in equilibrium; therefore p and q' are parallel to the plane of X Y. Then if d F denotes the area of the inclined face, the conditions of static equilibrium furnish the equations, *

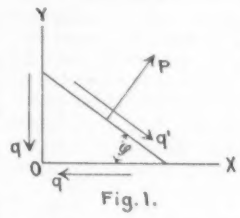


Fig. 1.

$$p\,dF\sin\varphi + q'\,dF\cos\varphi - q\,dF\cos\varphi = 0,$$
$$p\,dF\cos\varphi - q'\,dF\sin\varphi - q\,dF\sin\varphi = 0,$$

whence

$$p = q\sin 2\varphi,$$
$$q' = q\cos 2\varphi.$$

For a maximum or minimum value of p,

$$\frac{dp}{d\varphi} = 2q\cos 2\varphi = 0,$$

whence

$$\varphi = \frac{\pi}{4} + n\frac{\pi}{2},$$

where n is any positive integer. Therefore, on any plane of maximum or minimum normal stress,

$$q' = 0.$$

For the second differential of p with respect to φ, we have

$$\frac{d^2p}{d\varphi^2} = -4q\sin 2\varphi.$$

Consequently the planes of maximum and minimum normal stress are mutually perpendicular, and each inclined at an angle of 45° to the horizontal, the normal stresses on these planes being $p = +q$ and $p = -q$, respectively. The position of these planes for a beam uniformly loaded is illustrated in Fig. 2.

From (1) it is seen that for any given vertical section, q is greatest when u = o; that is to say, for any given vertical cross-section the intensity of the shearing stress is greatest for points on the neutral axis. It is also evident from (1) that for any point at a given distance u from the neutral axis, q is greatest where S is greatest. From these two considerations it follows that the intensity of the normal tensile or compressive stress on planes inclined at 45° to the horizontal is greatest for points on the neutral axis in the plane of maximum shear, and at any such point is equal to the vertical (or horizontal) shear at the same point.

In an article by Prof. Hatt on the "Theory of the Strength of Beams of Reinforced Concrete," published in Engineering News for Feb. 27, 1902, the shearing stresses were left entirely out of consideration. Thus Prof. Hatt's formulas are only useful in dimensioning beams to resist bending moment, while as a matter of fact the actual failure of beams is due in most cases to the tensile stresses arising from shear, as was forcibly pointed out by Capt. Sewell in an article published in Engineering News of Jan. 29, 1903.

The fact that cement exhibits high compressive, but low tensile, strength, necessitates, in cement-steel construction, that the steel be designed to carry all, or nearly all, of the tension. For the present discussion we will assume a beam of rectangular cross-section on an inverted steel T-bar as base. The horizontal leg of the T-bar will then act like the lower flange of a plate girder in resisting direct tensile stress, while the vertical leg will have an important effect in resisting the tensile stresses due to shear, as will appear later. The method here given is perfectly general and will apply to any form of section whatever.

Let b denote the breadth of the beam, and con-

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†Föppl: "Technische Mechanik," III., Festigkeitslehre, 55—56.

*Föppl: "Technische Mechanik," III., Festigkeitslehre, 58.

†Festschrift der techn. Hochschule in Brünn, 1899.

‡Föppl: "Technische Mechanik," III., Festigkeitslehre, 132-133.

§Ibid, p. 21.

sider a section of length d x, cut out by planes parallel to the plane of Y Z. Let N N′ denote the neutral axis, and d φ the angle of distortion (Fig. 3). Then the direct tensile and compressive stresses acting on A A′ B′ B can be divided into five parts; the compressive stresses F_1 and F_2, which are the resultants of the stresses acting on A N D E E′ D′ N′ A′ and D E E′ D′, respectively, and the tensile stresses F_3, F_4, F_5 and F_6, which are the resultants of the stresses acting on D G G′ D′, D′ N′ C′ G′, D N C G and B B′ C′ C, respectively. Let

E_s = Young's modulus for steel,
E_c = " " " cement,
p_s = intensity of fiber stress on steel,
p_c = " " " " " cement.

Then, if the origin of coördinates is taken in the neutral surface we have, since $\frac{y\,d\varphi}{dx} = \frac{p}{E}$,

$$p_s = E_s\, y \frac{d\varphi}{dx}. \qquad p_c = E_c\, y \frac{d\varphi}{dx}.$$

Consequently,

$$F_1 = \int p_c\, b\, d\, y = \int_o^e E_c\, b \frac{d\varphi}{dx}\, y\, dy - \int_o^{e-h+l} E_c\, t \frac{d\varphi}{dx}\, y\, dy =$$

$$E_c\, b \frac{d\varphi}{dx}\frac{e^2}{2} - E_c\, t \frac{d\varphi}{dx}\frac{(e-h+l)^2}{2}$$

$$F_2 = \int p_c\, t\, dy = \int_o^{e-h+l} E_c\, t \frac{d\varphi}{dx}\, y\, dy = E_c\, t \frac{d\varphi}{dx}\frac{(e-h+l)^2}{2}$$

$$F_3 = \int p_c\, t\, dy = \int_o^{h-e} E_c\, t \frac{d\varphi}{dx}\, y\, dy = E_c\, t \frac{d\varphi}{dx}\frac{(h-e)^2}{2},$$

$$2\,F_4 = 2\int p_c \frac{b-t}{2}\, dy = \int_o^{h-e} E_c\,(b-t)\frac{d\varphi}{dx}\, y\, dy =$$

$$E_c\,(b-t)\frac{d\varphi}{dx}\frac{(h-e)^2}{2},$$

$$F_5 = \int p_s\, b\, dy = \int_{h-e}^{h+k-e} E_s\, b \frac{d\varphi}{dx}\, y\, dy =$$

$$E_s\, b \frac{d\varphi}{dx}\left[\frac{(h+k-e)^2 - (h-e)^2}{2}\right]$$

The condition Σ hor. forces $= 0$ gives

$$F_1 + F_2 = F_3 + 2\,F_4 + F_5,$$

or,

$$E_c\, b \frac{d\varphi}{dx}\frac{e^2}{2} - E_c\, t \frac{d\varphi}{dx}\frac{(e-h+l)^2}{2} + E_s\, t \frac{d\varphi}{dx}\frac{(e-h+l)^2}{2}$$
$$= E_c\, t \frac{d\varphi}{dx}\frac{(h-e)^2}{2} + E_c\,(b-t)\frac{d\varphi}{dx}\frac{(h-e)^2}{2} +$$
$$E_s\, b \frac{d\varphi}{dx}\left[\frac{(h+k-e)^2 - (h-e)^2}{2}\right].$$

Dividing through by the constant $\frac{1}{2}\frac{d\varphi}{dx}$ and reducing, this becomes

$$(E_s - E_c)\, t\,(l^2 + 2\,e\,l - 2\,h\,l) = E_c\, b\,(h^2 - 2\,h\,e) + E_s\, b\,(k^2 + 2\,h\,k - 2\,k\,e)$$

whence

$$(2)\quad e = \frac{E_c\, b\, h^2 + E_s\, b\,(k^2 + 2\,h\,k) + (E_s - E_c)\, t\,(2\,h\,l - l^2)}{2\,l\,t\,(E_s - E_c) + 2\,b\,h\,E_c + 2\,b\,k\,E_s}$$

which gives the distance of the neutral axis below the upper surface of the beam.

If the cement is assumed to carry all the compressive stress but no tensile stress, we must put $e = h$ in (2), in which case this formula becomes,

$$(3)\quad h^2 = k^2 \frac{E_s}{E_c} - \frac{l^2\, t\,(E_s - E_c)}{b\, E_c}$$

This gives a relation between h and k from which either can be found when the other is known or assumed.

If the material of the beam were homogeneous throughout we would have $E_s = E_c$, and in this case (2) becomes

$$e = \frac{b\,(h^2 + 2\,h\,k + k^2)}{2\,b\,(h+k)} = \frac{h+k}{2}$$

that is to say, the neutral axis would pass through the center of the section.

For homogeneous beams the moment of resistance of any vertical cross-section is given by the formula

$$M = \frac{p\,I}{e}$$

where p = intensity of stress on extreme fiber, e = distance of extreme fiber from the neutral axis, and I = moment of inertia of the section about the neutral axis. In the present case the moment of resistance of the beam is the sum of the moments of the separate forces F_1, F_2, F_3, F_4 and F_5 about the neutral axis. Instead of finding these moments separately, however, we may proceed as follows: If p_c and p_s are the intensities of the stresses acting on a particle of cement and steel, respectively, at the same distance from the neutral axis, then

$$E_s\, p_c = E_c\, p_s, \quad \text{or} \quad p_s = \frac{E_s}{E_c}\, p_c.$$

Therefore if d A is an infinitesimal area of steel at the distance y from the neutral axis and the moment of the stress acting on this area is taken over any given portion of the section, we have

$$\int p_s\, d\, A\, y = \int \frac{E_s}{E_c}\, p_c\, d\, A\, y.$$

Consequently, the intensity of the fiber stress can be considered to vary directly as its distance from the neutral axis over the whole vertical section of the beam, provided the area of the steel is increased in the ratio of $\frac{E_s}{E_c}$. If, then, the depth is kept constant, the breadth must be increased in this ratio. Therefore, to find the moment of resistance of the beam at any section, divide the section into three parts; A, of depth $(h-l)$ and width b; B, of depth l and width $(b-t) + t\frac{E_s}{E_c}$; and C, of depth k and width $b\frac{E_s}{E_c}$, as illustrated in Fig. 4.

Then the moment of resistance of the beam at

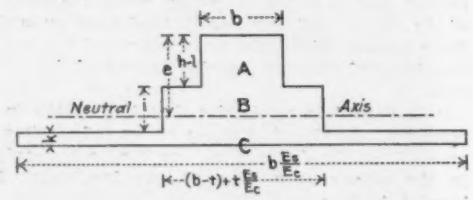


Fig. 4.

any section is given by the ordinary bending formula,

$$(4)\quad M = \frac{p\,I}{e}$$

where p = intensity of normal stress on extreme fiber of *cement*, e = distance of this fiber from the neutral axis, and I = moment of inertia of the section of dimensions given in Fig. 4 about the neutral axis. If p′ = intensity of normal stress on extreme fiber of *steel*, we have,

$$(5)\quad p' = \frac{E_s}{E_c}\, p = \frac{E_s\, M\, e}{E_c\, I}$$

For such materials as steel, wrought iron and wood it can be easily proved that if a beam of rectangular cross-section and of length greater than its breadth is so designed as to be safe against bending moment, its safety against shear is assured. In the present case, however, it is necessary to determine first, whether or not the shear in the horizontal plane of junction between the cement and steel is sufficiently great to overcome the adhesion between cement and steel, and second, whether or not the intensity of the normal tensile stress on planes, inclined at 45° to the horizontal, is within the limit of safe tensile stress for cement.

Let E_c' and E_s' denote the moduli of shearing elasticity for cement and steel, respectively, and let q_c and q_s denote the intensity of shear at any point on cement and steel, respectively. Then as a result of our elementary assumptions that in a concrete-steel construction the concrete and steel act together as one substance, and that sections which are plane before flexure remain plane after flexure, we have

$$\frac{q_c}{q_s} = \frac{E_c'}{E_s'}$$

Upon the same assumptions as above, if p_c and p_s denote the intensity of the normal stress on cement and steel, respectively, at any point in a plane section inclined at 45° to the horizontal, we have

$$\frac{p_c}{p_s} = \frac{E_c}{E_s}.$$

Since the normal stress at any point in such an inclined plane is equal to the vertical or horizontal shear acting at the same point, this necessitates the condition,

$$\frac{E_c'}{E_s'} = \frac{E_c}{E_s},$$

a result capable of rigorous mathematical demonstration,* and which is also confirmed by experiment.

Now take a horizontal longitudinal section of the beam at an infinitesimal distance above the horizontal flange of the T-bar. Then the area of steel in a horizontal strip of width dx in this plane is t dx, and the area of cement is $(b-t)$ dx. Consequently, if q is the intensity of the shear on this strip, we have,

$$q\, b\, dx = q_c\,(b-t)\, dx + q_s\, t\, dx$$

whence

$$(6)\quad q_c = \frac{q\, b\, E_c'}{E_c'\,(b-t) + E_s'\, t}$$

If the greatest value of q_c obtained from this expression is less than about 550 lbs. per sq in., which is the safe adhesive strength of cement and steel, the beam is safe as regards the slipping of the cement on the steel.

The only other section of the beam that need be considered is a section by a plane parallel to the axis of z and inclined at an angle of 45° to the horizontal. As above, the area of steel in a strip of width dx $\sqrt{2}$ and extending across the beam is t dx $\sqrt{2}$, and the area of cement in this strip is $(b-t)$ dx $\sqrt{2}$. Therefore, if p_c and p_s are the intensities of the normal stresses for cement and steel, respectively, we have,

$$q\, b\, dx\, \sqrt{2} = p_c\,(b-t)\, dx\, \sqrt{2} + p_s\, t\, dx\, \sqrt{2}$$

whence

$$(7)\quad p_s = \frac{q\, b\, E_c}{E_c\,(b-t) + E_s\, t}.$$

It is to be remembered that q is greatest at the neutral axis in the plane of maximum shear. The value of p_c should be calculated for this maximum value of q and compared with the value of p_c in a horizontal section just above the vertical leg of the T-bar for which $t = 0$, $E_c = E_s$, and consequently $p_c = q$. The greater of these two values should then be less than the maximum allowable tensile stress for cement.

A more advantageous distribution of material may perhaps be secured by dispensing with the vertical leg of the T-bar, using simply a horizontal plate at the bottom of the beam, stirrups being used to insure safety against tensile stress. For this case $t = 0$ and formulæ (2), (3), (6) and (7) become,

$$(2')\quad e = \frac{E_c\, h^2 + E_s\,(k^2 + 2\,h\,k)}{2\,h\,E_c + 2\,k\,E_s},$$

$$(3')\quad h^2 = k^2 \frac{E_s}{E_c},$$

$$(6')\quad q_c = q,$$

$$(7')\quad p_s = q,$$

while (4) and (5) remain the same, the I for this case being the moment of inertia of the section shown in Fig. 4, in which the part marked B is now of width b instead of $(b\text{-}t) + t\frac{E_s}{E_c}$, as before.

From the foregoing considerations and Fig. 2 it is apparent that the stirrups should not be placed vertically, but perpendicular, to the planes

*Johnson: "Materials of Construction, p. 9.

of maximum tensile stress, and therefore inclined at an angle of 45° to the horizontal. A simple form of stirrup and attachment is shown in Fig. 5.

Whatever the form of stirrup used, the area of cross-section of metal required may be found as follows: The maximum intensity of tensile stress is first found for points midway between successive stirrups. The average of these two intensities is taken and multiplied by the width of the beam times the distance between two successive stirrups measured on a horizontal line. (This latter should not be greater than the depth of the beam.) If this tension is divided by the safe unit tensile stress for steel it will give the area of the steel required in a cross-section of the stirrups.

SOME RECOMMENDATIONS CONCERNING ELECTRICAL AND MECHANICAL SPECIFICATIONS OF TROLLEY-LINE INSULATORS.*

By Samuel Sheldon† and John D. Keiley.‡

At present there appears to be no standard basis for comparison of the relative merits of insulators which are supplied for use in ordinary overhead trolley line construction. It is obviously desirable that there should be a definite basis upon which a proper selection can be made.

With a view to formulating specifications of such materials, it was found necessary, on account of the lack of published data on the subject, to conduct a series of tests to determine the electrical and mechanical properties of these materials. The tests were made upon samples obtained in the open market from the stocks of well-known manufacturers. Some of the results of these tests are given below and recommendations concerning specifications of certain types are appended.

Determinations were made of the tensile strength of the samples, of the voltage necessary to perforate the insulation or arc between conducting parts, of the mechanical softening temperature, and of the relative magnitudes of the insulation resistances.

TENSILE STRENGTH.—These tests were made in the usual manner, the samples being pulled apart by means of a Riehlé 30,000-lb. universal machine. U-shaped pieces of round steel were passed through the metal eyes of such samples as globe, Brooklyn strain, and terminal strain insulators, and were clamped in the jaws of the machine. The insulated bolts were passed through a proper-sized hole in a steel plate. The threaded end was connected with a steel eyebolt by means of a steel threaded sleeve, and the tension was exerted between the eyebolt and the plate. The results obtained from breaking the samples are given in Table I., the product of different manufacturers being represented by the letters A. B, C and D. The numerals represent the tension in pounds at the time of break.

TABLE I.—Tensile Tests of Insulators.

2½-in. Globes.

A	B	C	D
3710	5950	7725	6320
4510	5770	7475	6890

3-in. Globes.

A	B	C	D
4210	11[illegible]0	5450	....
5310	8930	5550	....

Small Brooklyns.

A	B	C	D
9990	....	10320	5520
11130	....	9010	6450

Large Brooklyns.

A	B	C	D
11490	19670	18510	....
10510	17140	18250	....

Insulated Bolts.

A	B	C	D
6379	6130	7360	3280
4495	5490	9010	2925

Some samples gave way in the eyes, others broke in the insulation. It is very desirable that strain insulators should be so designed that when subjected to a test for tensile strength, they should give way in the eyes. As these insulators are subjected to an uncertain strain while being installed and after being in use, it is of great importance that one may be able to depend upon the fact that, if the eyes be intact, the insulation is also in good condition.

BREAKDOWN VOLTAGE.—In carrying out these tests, the two metal portions or conductors of the insulators were connected respectively with the two high pressure terminals of a 1:200 step-up transformer, whose low pressure terminals were connected in series with a regulating rheostat to a supply of alternating current at a frequency of 60. A dead-beat voltmeter was connected to the low pressure terminals. By manipulating the rheostat any desired voltage could be impressed upon the low-pressure coil and its value could be determined from the voltmeter. The high pressure voltage would be 200 times as large provided the secondary was unloaded, that is provided the insulator were unpunctured and there were no arcing present. The test was started with a low impressed voltage which was gradually raised until a sudden drop in the voltmeter reading indicated that a breakdown had occurred. The maximum reading multiplied by 200 is the breakdown voltage. Table II. summarizes the results obtained.

TABLE II.—High Voltage Tests of Insulators.

2½-in. Globes.

A	B	C	D
8010	8610	5610	7500
....	7610	5110	6110
....	....	4510*	6910

3-in. Globes.

A	B	C	D
11410	12810	8010	....
10810	10810	9010	....

Small Brooklyns.

A	B	C	D
....	14810	6710	35000
....	....	5510*	....

Large Brooklyns.

A	B	C	D
8010	14410	11410	....
4210	13010*	7610*	....
4010*	....	....	....

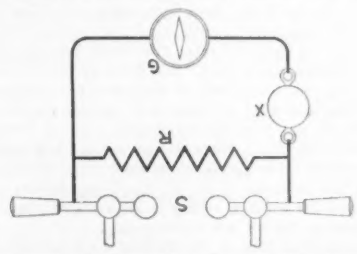
Insulated Bolts.

A	B	C	D
12010	11450	12210	arc at
13010	14450	10010	25000

Note.—Figures followed by an asterisk were obtained from tests on insulators which were, at the time, under a tensile strain of about 4,500 lbs.

The voltage necessary to rupture the dielectric would undoubtedly have been less than the amounts given in the table if the duration of the application of the high pressure had been increased. In practice, trolley insulators are seldom subjected to a voltage greater than that of the generators or converters. Occasionally, however, they are subjected to moderately high voltages of short duration which arise from the disappearance of magnetic flux that has been created by short circuits. As will be noted from the table, the tests made on insulators under simultaneous mechanical strain show that the dielectric strength is slightly reduced by strain, but in all the samples tested it was sufficiently high to meet the requirements of present practice.

HEAT TESTS.—Oftentimes when a trolley wire breaks it becomes heated on account of the grounding of a broken



Arrangement of Apparatus for Testing Insulation Resistance of Trolley-Line Insulators.

end. Sometimes the insulating material in the round-top hangers softens under the influence of the heat communicated from the wire and allows the ear and suspended wire to drop to the ground. Efforts were made to determine the temperature of the insulation at the time of softening. A hanger was screwed into a regular ear, and was suspended in an inverted position in a double walled oven. A wire was attached to the suspended cap and was passed through a hole in the bottom of the oven. To this wire was attached a weight of 100 lbs. This weight is equal to that of about 200 ft. of No. 000 copper trolley wire and it is quite possible that a hanger might be subjected to this strain. The temperature of the oven was raised by means of burning an arc lamp which was inclosed in it. As the temperature rose to a certain point, the insulating material softened and the suspended weight pulled the round top cap of the hanger away from its bolt. This temperature was noted on a mercury thermometer whose bulb was placed near the ear. Unquestionably, the temperature varies very much at different points inside the insulating material, such material being a good heat insulator as well as an electrical insulator. The material next to the metal, however, is probably the hottest and its temperature is nearly that of the metal. It is at this point that it softens most. The results obtained from three samples were, A 168° C.; B 168° C.; and C 145° C.

Appreciating the uncertainty of such a test, another method was devised which tested the hangers under working conditions. This method gives but relative results. Under the circumstances they are more to be desired than absolute values of temperature.

A soft iron round rod, ¼-in. in diameter and 20 ins. long, was clamped by an ordinary trolley ear, whose ends had been cut off so as to leave a length of 5½ ins. This was suspended by means of a wire stirrup underneath a beam. Into the ear was screwed the bolt of the round top hanger to be tested. To the cap of the hanger was suspended a weight of 200 lbs. A current of 200 amperes was then sent through the iron. The time which elapsed between closing the circuit and the separation of the parts of the hanger under the influence of the weight was noted.

These times in minutes were as follows:

A	B	C	D
50	34	94	74

Two similar samples from the same manufacturer were tested, one with the cap underneath, as above described, and the other with the ear beneath. The times between the closing of the circuit and the breaking down of the insulators were respectively 31 and 33 minutes. This indicates that the position in regard to vertical arrangement during test is immaterial.

Efforts were made to determine roughly the character and the composition of the insulating materials employed by each manufacturer. Upon ignition, by applying a lighted match, each of the insulating materials burned quietly with a very small flame. There was some smoke and in all cases the characteristic odor of burning shellac. Each sample when placed in alcohol went partially into solution, leaving a residue. Mica and asbestos were present in some of the residues.

If the binding material be, in all cases, shellac, it is evident that the softening temperature is an indeterminate quantity. Shellac is somewhat viscous at ordinary temperatures, and its viscosity rises rapidly with increase of temperature. Insulating bolts, one from each manufacturer, were placed in boiling water and were allowed to remain until they had assumed the temperature of the water, i. e., 100° C. In each case the insulating material had softened so as to permit of molding under slight pressure. The viscosity, at a given temperature, is also dependent upon the relative amount of shellac to the other material present. This amount is liable to variation in samples of the same type from the same manufacturer.

INSULATION RESISTANCE.—The resistance of an ordinary strain insulator is very large, and, if it were not for the large number of them which are connected in parallel on a trolley system, no consideration need be given to this point. It is difficult to determine rapidly the individual resistances in ohms, hence the following method of determining the relative resistance values was devised.

A Holtz machine when run at a constant speed, owing to its practically infinite internal resistance, functionates as a constant current generator. For obtaining the comparative values of the resistances of the insulators, such a machine was used as a source of e.m.f., and an arrangement of apparatus as shown in Fig. 1 was employed, in which S is the spark gap of the Holtz machine, X is the insulator under test, G is a mirror galvanometer, and R is a resistance in which is included the leakage resistance of the Holtz machine, and of the connecting wires.

By connecting together the two terminals of the insulator X, by means of a copper wire, practically all of the current I, which is produced by the machine, is made to pass through the galvanometer and it produces a deflection θ. Upon removing the copper wire, a current i_x will flow through the galvanometer, giving a deflection θ_x.

This deflection will be inversely proportional to the resistance of X. The rest of the constant current I will pass through R, which includes the leakage paths. Representing this current by i_r, there exists the relation,

$$X = \frac{I - i_x}{i_x} R,$$

or, since I and i_x are proportional to deflections θ and and θ_x respectively,

$$X = \frac{\theta - \theta_x}{\theta_x} R.$$

By substituting for X another insulator, its resistance can be quickly determined in terms of R in a similar manner. This method is better suited for getting the comparative resistances of insulators of the same shape, than for getting their absolute resistance. A portion of an insulator's conductance unquestionably consists of surface leakage.

The resistances obtained are given in Table III. in terms of an arbitrary standard:

TABLE III.—Insulation Resistances of Trolley-Line Insulators.

Globes, 2-in.			Globes, 2½-in.			Brooklyns, Small.		Brooklyns, Large.		
A	B	C	A	B	C	B	C	A	B	C
61	98	1.7	27	36	1	35	40	0.7	39	27
55	76	12.0	55	18	1.5	35	43	0.8	41	27
			61	37	1					

SPECIFICATIONS.

Specifications for the various forms of insulators used in trolley construction must vary with local conditions and with the policy of the user. Specifications for Globe and Brooklyn Strain insulators should cover the following points:

1. Dimensions.
2. Size of eye.
3. All samples tested shall break in the eye.
4. The average ultimate tensile strength of all samples subjected to mechanical test shall not be less than A lbs., and no individual sample shall show a tensile strength of less than 85% of the average tensile strength of all the samples that are tested.

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5. The average breakdown voltage, for samples which have been broken in the eye in the mechanical test, shall not be less than B volts, and no individual sample shall break down at less than 90% of B volts.

As to the values to be specified for ultimate tensile strength and breakdown voltage, the following are suggested, where high-class insulators for use on 500-volt lines are to be specified.

	A Ultimate Tensile Strength in lbs.	B Breakdown Voltage.
2½-in. globes	6,000	7,000
3-in. globes	9,000	10,000
Small Brooklyns	9,000	10,000
Large Brooklyns	18,000	10,000

Owing to the comparatively low softening temperature of the insulating materials generally used, and to the close proximity of the working conductor when in service, it is important that specifications for round-top hangers should impose a test for softening temperature. The following "hot rod test" is suggested:

Round-top hangers when suspended free from draft in an inverted position by means of a bronze ear weighing 8 oz. and being 5½ ins. long, the ear clamping the middle of a round rod of soft iron ¼-in. in diameter and at least 20 ins. long, between connectors, must be able, without breaking down or becoming permanently deformed by more than 1-16-in., to sustain a weight of 200 lbs. from the cap for one hour, a current of 200 amperes being passed continuously through the iron rod, the rod being cold at the start.

RIVETS IN STRUCTURAL STEEL WORK.*

By C. J. Tilden,† Assoc. M. Am. Soc. C. E.

A theoretically perfect rivet should fill the hole completely, be of homogeneous material throughout, and have two well-formed heads. The strength of a riveted joint depends, theoretically, on but two considerations: first, the shearing strength of the rivet material, usually soft steel; and, second, the number of rivets used. When comparatively thin plates are joined by rivets of large diameter, it may happen that the resistance of the metal to crushing is less than the shearing strength of one rivet; in which case the crushing or "bearing" value of the metal determines the value to be given to each rivet in calculating the strength of the joint. The question then arises, with what degree of safety may the designing engineer accept these theoretical assumptions, and how are they borne out by the conditions which occur in shop practice?

In the first place, the material of a rivet is not homogeneous. In a large majority of cases, it is probable that test pieces taken from different parts of a rivet after driving, assuming that such small pieces could be properly tested, would show widely different characteristics, and these totally different from similar tests of the same rivet before driving. A very good idea of the great difference in quality of rivet material after driving may be gained by watching for a few hours a shop gang engaged in cutting out rivets which have been condemned by the inspector. Sometimes the metal is hard, tough, and fibrous; then again nearly as soft, to all appearances, as lead or pewter; and occasionally the rivet head will fly off at the first blow of the hammer, apparently almost as hard and brittle as glass.

A second noteworthy discrepancy in the design of riveted joints is the failure to take account of the action of the rivet heads in bringing the two or more surfaces into very close contact, so that a large amount of friction is developed. It is quite possible that this friction may amount to more than the shearing strength of the rivet. In any event, it is a very important factor in the strength of a riveted joint.

In the diagram, Fig. 1, are shown some of the more frequent imperfections in rivet work, resulting from carelessness of the workmen. At a, for comparison, is sketched a perfectly driven rivet. The original form is shown dotted, the "shank" being 1-16 or 3-32 of an inch less in diameter than the hole which it is to fill, and enough longer than the "grip" or length between heads, to allow the formation of the new head, and the squeezing out of the rivet material sufficiently to fill the hole completely. Both heads should be concentric with the shank, and the rivet should be perfectly tight, giving a clear, sharp ring when struck with a light hammer.

At b is shown a loose rivet which has been "calked," with a cold-chisel, to make it appear tight under the inspector's hammer—a favorite trick of careless riveting gangs, and often very difficult to detect; if suspected, a close examination should be made of the head of the rivet for signs of the calking-tool, especially if the rivet has been generously bespattered with fresh paint or tobacco juice. Both these commodities, always plentiful in the shop, are favorite means of concealment for "scamped" work of this character. A result very similar to calking, but much harder to discover, is sometimes secured by using the riveting machine, or "bull," as it is familiarly known to the shop men, on the cold rivet. The movable cup of the "bull" is brought sharply against the rivet head, securing somewhat the effect of a blow, and this is repeated four or five times on each loose rivet. In general, this machine calking is not very effective, but the writer has known instances where it has been successful. It is well-nigh impossible to tell from the appearance of the rivet head afterwards if this trick has been attempted. A very slight polish on the head of the rivet is about all the evidence that ever appears, and this is readily hidden by a dab of grease or dirt, or the ever-ready tobacco juice. It is a form of "scamping" that is seldom resorted to, however, as it is more work than calking with the cold chisel, and far less likely to accomplish its purpose.

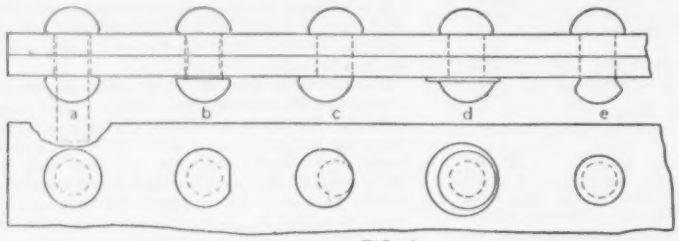


FIG. 1.

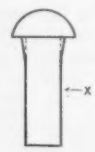


FIG. 2.

The sketch, b, also shows the probable result of heating the rivet unevenly. Where the heating is done in an ordinary portable forge, fired with coke, the forge tender gets into the habit of heating only that part of the rivet which is to be upset to form the head, leaving the remainder comparatively cool. Referring to Fig. 2, for example, from the lower end of the rivet to, perhaps, the point x, the metal is at white heat; above that it cools rapidly until the head is practically "cold," often not even a dull red color. This uneven heating not only prevents the rivet from upsetting throughout its length, and so filling the hole, but is apt to injure the quality of the metal above the point x, owing to its being worked under the hammer at too low a temperature.

Careless manipulation of the riveting machine may result in the condition shown at c, where the head is not concentric with the shank. The fault can be detected only by comparison with the other rivets in the joint, showing uneven spacing and irregular lines.

The condition shown at d results from too much metal in the shank of the rivet before driving, giving a "soldier-cap" head. The reverse of this is shown at e.

It must not be supposed that these defects are the only ones which occur in rivet work; they are only a few of the more frequent errors of this kind that may be observed in any shop. Combinations of two or more of the forms shown occur not infrequently, and an almost endless variety of changes may be rung on each one. Of the four types, b and e should be condemned unquestionably whenever found, being not only bad workmanship but unreliable. c and d probably develop the full strength of the rivet, and may be allowed to pass if strength is the only consideration; but if the work is to be exposed they should be cut out and replaced, as they are sure to look ragged in finished work.

As to the actual difference in strength between a perfect rivet, as a, and any of the imperfect ones, it is impossible to judge with any degree of accuracy. In fact, if a test were made it is quite conceivable that a rivet such as b, or even e, might develop greater strength than a. About all that can be said is that this is not likely to happen, but rather the reverse, as a properly driven rivet is more likely to develop its full strength than one which is imperfect in any way. But this is not reducing the question to any scientific basis, and, indeed, it cannot be so reduced. Rigid specifications are required for riveted work, and the work in the shop is subjected to the most careful inspection, not because a carelessly driven rivet is less strong, by any definitely calculable percentage, than one which is properly driven, but for the simple reason that careful and accurate work is more reliable.

The nearest approach to a theoretically perfect rivet is probably the turned bolt which is occasionally used for field connections. In such cases it is the practice of some engineers to require the holes to be drilled instead of punched, or "sub-punched and reamed"—that is, punched to a diameter about ¼-in. less than that of the bolt to be used and reamed to proper size. The bolt is turned to a driving fit, and the threaded part is of slightly reduced diameter, the shoulder, s, protecting the thread while the bolt is driven home. To keep the nut in place after it is screwed up tight, the projecting threaded end of the bolt is upset against the nut. In spite of the reliability of this connection, however, its high cost precludes its general use.

Fig. 2 shows a form of rivet which has certain advantages and disadvantages over the ordinary shape. In this form the shank is slightly increased in diameter (exaggerated in the drawing) for a distance of ⅛ to ¾-in. from the head. Directly under the head, at the base of the cone-like enlargement, the shank has the same diameter as the hole into which the rivet is to go—that is, from 1-16 to 3-32-in. larger than the main part of the shank. This is an advantage in the shop, where the rivet is sure to be uniformly heated throughout its length, as it insures the complete filling of the hole up to the rivet head. In the field, however, where the rivets are likely to be unevenly heated, such a design would be of doubtful advantage. A rivet of this shape might easily appear sound and tight under the inspector's hammer, and yet have been very imperfectly driven.

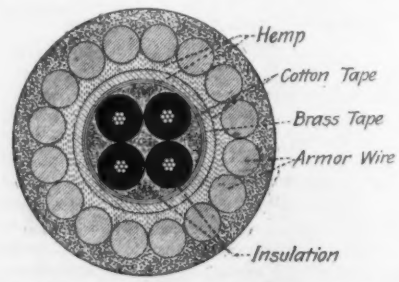
*Condensed from an article in The Harvard Engineering Journal."

†Assistant Engineer New York Rapid Transit Commission, 242 St. Nicholas Ave., New York City.

THE LONGEST SUBMARINE TELEPHONE CABLE.

A few weeks ago a telephone line between England and Belgium was opened for public use. The line from London to Brussels is 244 miles long, of which the overhead line in Belgium is 93 miles and that in England 97 miles. The submarine cable connecting the two land lines runs from St. Margaret's Bay, England, to La Panne, Belgium, and has a total length of 54 miles (47.3 nautical miles). It is thus the longest submarine telephone cable yet put into operation. As a matter of general interest we show the accompanying drawing of the cross-section of this cable, reproduced from "The Electrician" (London). The cable was made by Henley's Telegraph Works Co., of England.

The cable is about 4⅛ ins. in diameter, weighs 4.9 lbs. per ft., and contains four separate conductors, giving two metallic circuits. Each conductor is made up of seven wires laid up in a left-hand strand. The four strands are separately insulated with alternate layers of gutta-percha and Chatterton compound, and are then wound around a core of tarred hemp. After rounding off this central section with tarred hemp, a wrapping of prepared cotton tape was put on, then a layer of brass tape. This brass sheathing is to protect the cable from the teredo; the brass strip is 1½ ins. wide by 0.04-in. thick. Over the brass taping comes another layer of cotton tape and a layer of tarred hemp. The hemp provides the bedding for the wire armor, which consists of sixteen ¼-in. galvanized iron wires coated with tar. A final wrapping of two layers of hemp, in reverse lay, covers the armor.



Cross-Section of Submarine Cable on England-Belgium Telephone Line.

The specifications for the cable required, among other things, that the resistance of each (7-wire) conductor should not exceed 4 ohms per kilometer, or 6.4 ohms per mile. The following test treatment was given to each of the four insulated conductors before they were laid up over the hemp core: The conductor, coiled up, was placed in a tank of water for two weeks; then the water was heated to 75° F. for 24 hours, at the end of which period an alternating current of 100 cycles at 5,000 volts pressure was applied for 15 minutes. After this the resistance of the insulation and the capacity of the cable were measured, using a battery giving 300 volts. The requirements were: Insulation resistance not less than 500 megohms per mile, and capacity not to exceed 0.275 micro-

farads per mile. At the joints of the several sections the insulation resistance was required to be at least half as great as at other parts. The cost of the cable was about $2,900 per mile, at the makers' works.

A GRAPHICAL METHOD OF SETTING SLOPE STAKES.

By J. M. Rudiger.*

In setting side stakes for work in excavation and embankment, only one way of finding the desired intersection point for the stake ordinarily suggests itself; that is, by trial. However, other ways have been devised, and the following, while perhaps not so quick as some others, is nevertheless so simple and effective that it is submitted in the belief that if tried it will help to turn work into mere play.

When in the field my "topog box," with paper, scale dividers, protractor, etc., is usually hanging by its strap from my shoulder or neck, and serves ont only as a catch-all and carry-all, but also as a writing and drawing table, always ready and easily adjustable.

For staking out, a cross-section sheet is prepared as in the accompanying sketch,† and is pinned on the free side of the box.

The preparation of this sheet consists in drawing lines $A_1 A_2 A_3$ and $B_1 B_2 B_3$, representing, respectively, one-half section of embankment with a slope of 1½ to 1 and of excavation with a slope of 1 to 1, or otherwise as desired. The line $A_1 B_1$ is center line of the section. The horizontal lines are numbered to indicate feet of cut or fill, tenths being shown by the intermediate divisions. Likewise distances out from the center line, $A_1 B_1$, are shown by the vertical lines and divisions.

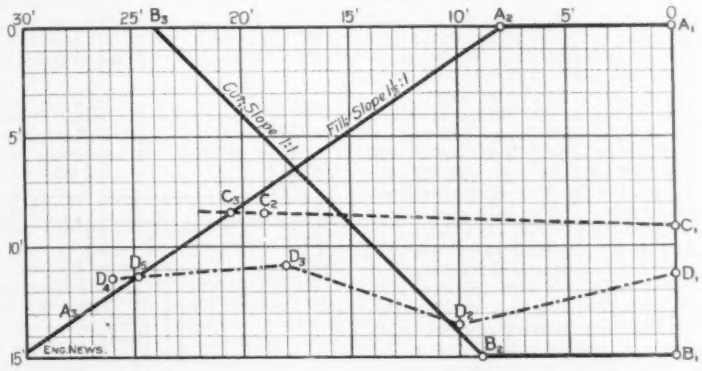


FIG. 1. DIAGRAM FOR SETTING SLOPE STAKES.

Along with this cross-section sheet there are required two pins, with a piece of fine silk thread made fast to one of the pins, which may be carried stuck in the lapel of the coat.

Proceeding to set stakes at any station, the center cut or fill is known from the record book; and at that height on the sheet, say, for instance,

fill $\frac{-9.1}{00}$ the pin with thread attached is stuck on center line at — 9.1 (at C_1), then if the ground is seen to be nearly a uniform surface and slightly higher on one side than at the center line, make a guess as to the probable position of the intersection of the side slope with the ground, at C_2, say, 19 ft. out; hold the rod at C_2 and read the elevation with the level, which, let us assume, gives a fill of $\frac{-8.5}{19.0}$. When this reading is called out by the leveler the thread is stretched from C_1 through this point C_2 on the sheet, holding the thread so its intersection with the slope line $A_2 A_3$ is shown at C_3. This shows at once that C_3 is $\frac{-8.5}{20.7}$, and gives the correct position of the stake graphically. Record of this point is entered in the note book, and the distance 20.7 ft. is called out to the stakeman as the point at which to drive the stake.

In case of irregular ground, as in D_1 D_2 D_3 D_4, with center fill at D_1 of $\frac{-11.3}{00}$, the rod is sent to the first break in the ground at D_2, and a reading is taken showing a fill of — 13.5 at a measured distance of ($D_1 D_2$) of 100 ft. and is noted $\frac{-13.5}{10.0}$. The next break at D_3 is likewise measured, and fill calculated, as $\frac{-10.8}{18.0}$, and so noted. This point, D_3, on the sheet is seen to be near the point for the stake; stick a pin in the sheet at D_3 and send the rod out to the next break at D_4, say 26.0 ft. from the center line, where the fill is found to be — 11.2.

Now stretch the thread over the pin at D_3 and across to D_4, and its intersection with the "slope line" at D_5 is seen to be at $\frac{-11.1}{24.7}$. Note this in the record book, and call out the distance 24.7 ft. for the stake to be driven. You are then ready to find the point for the stake on the other side of the center line, using the cross-section sheet in exactly the same manner.

The leveler needs only to note the rod readings, the elevations and the cut or fill. Whoever is making the section notes records the cut or fill together with the distance from center line, as the measurements are made, and has constantly before him on the board the representation of the section, so that without trial computation or liability of confusion, by using the thread, he sees just where the ground line intersects the slope line and reads directly the true cut or fill and the distance from center line to the point where the stake belongs.

*114 South Elliot Place, Brooklyn, N. Y.

†In the sketch the lines showing tenths of a foot are omitted, but in practice they must be on the paper, and on a larger scale, of course, than in the sketch.

EFFECT OF FINENESS OF GRAIN ON THE FUSIBILITY OF CLAY.*

By Heinrich Ries,† M. Am. Inst. M. E.

The fusion-point of clay has usually been considered to be influenced chiefly by the quantity of fluxing impurities which the clay contained. Those clays which contain a high percentage of fluxes, such as ferric oxide, lime, magnesia, and alkalies, fuse at a low temperature.

Aside from the percentage of fluxing-impurities, the size of the grains exerts an important influence. Yet, while this fact has, no doubt, been recognized by many, practically no attempts have been made to prove it by actual experiment, although in the case of fire-clays, Prof. H. O. Hofman has rather disputed it.

To illustrate the point more clearly, suppose a clay to be made up of a mixture of very refractory grains, such as kaolinite, and some fusible ones. If this clay be heated, the more easily fusible grains will melt first on the surface, and the softening under heat will gradually extend from the surface of the grain to its center. The larger, therefore, the size of the grain, the longer the time required for the heat to penetrate it. Now, if two samples of clay contain the same volume of flux, but of unequal size, the one containing the smaller-sized grains should fuse first, because the fluxing materials are more evenly and thoroughly distributed through the clay, so that they will more or less surround the refractory grains, which tend to act as a framework and hold up the clay-mass when heated.

*A paper read at the Albany meeting of the American Institute of Mining Engineers, February, 1903.

†Instructor in Economic Geology, Cornell University, Ithaca, N. Y.

In order to demonstrate the effect of fineness of grain on the fusibility of clay, several mixtures were made up, consisting of white clay and other minerals. The white clay used was a very refractory type from Georgia, the fusion-point of which lay about Cone 35 of the Seger series. The mixtures were formed into small bars 1¼ ins. long, and set on a narrow surface, so that the ends were free to bend as soon as the mass softened. They were burned in a Seger gas-furnace.

One set of mixtures consisted of the following:

A. Equal parts of clay and hornblende, the latter being ground to pass through 150-mesh.

B. The same as A, but the hornblende passed through a 100-mesh screen and collected on 150.

C. The same as A, but the hornblende collected on 100-mesh screen.

When burned to Cone 5, the three were little affected, except that the one with the finest grains was colored uniformly red, while that with the coarsest presented a speckled appearance. When burned to Cone 8, the bar of mixture A was considerably bent at both ends, while that of B was nearly straight, and C was perfectly straight. At Cone 10, B was thoroughly fused, and C slightly bent.

This seemed to show well the effect of grain-size in the case of hornblende. The object of taking such a large amount of fluxing-material was simply to get results at moderate temperatures.

A second set of mixtures was then made up, consisting of equal parts of clay and calcite. The results obtained were similar to those found in the case of hornblende, but occurred at higher temperatures, since the calcite is a less powerful flux.

The mixtures were as follows:

D. Equal parts of clay and calcite, the latter ground to pass through 150-mesh.

E. The same as D; calcite passed through 100-mesh and collected on 150-mesh.

F. The same as D; calcite collected on 80-mesh.

These three, when burned to Cone 10, gave the following:

D. Bar fused, and bent, but not viscous.

E. Bar well vitrified, slightly bent, and edges sharp.

F. Bar straight, with but little evidence of fusion.

It appears quite clearly, therefore, that the size of grain exerts a marked influence on the point of fusion of the clay, and it would seem reasonable also to look for similar effects from other minerals. The writer has a series of experiments under way, the results of which will be presented later.

AIR HEATING WITH RADIATORS AND A CLOSED PIPING SYSTEM.

The most serious defect of the ordinary air-furnace system of heating is the irregularity and unreliability of its action. Everyone who has had dealings with an air furnace knows how difficult it is, even impossible in aggravated cases, to heat the windward side of a house when a steady, cold winter wind is blowing. If the circulation of the air is made more positive by confining the currents of heated air in a closed piping system we may expect to avoid this irregularity, but other troubles take the place of this one; the low specific heat of air necessitates more rapid circulation than in case of steam or water, and therefore would seem to require radiating surface of a different character and larger in amount.

What appears to be a successful application of closed-piping air heating was described by Mr. G. M. Aylsworth, of Collingwood, Canada, before the American Society of Heating and Ventilating Engineers at Niagara Falls, a week ago. There is the promise of practical success and commercial applicability in the method used, or rather in the form of radiator used, as this is the essential novelty of the experiment. We abstract the following description from Mr. Aylsworth's paper:

The general appearance and construction of the radiator are shown in Fig. 1. The conditions requiring to be met were: A distributing agent making the circuit of the furnace and radiators many times while water was making its circuit once; at the same time, since the rapidly moving air would absorb much less heat from the furnace, bulk for bulk, than the slower-moving water, the problem was to evolve a radiator that would compel the air passing through it to part with all or nearly all of its heat to the air of a room without diminishing the rapidity of travel of the confined air.

The problem has been solved by the adoption of four devices: (1) By increasing the distance the air has to travel within the radiator. (2) By offering the least possible obstruction to the passage of the heat from the air within the radiator to the air outside the radiator, but within the room; this was accomplished by using sheet

metal as thin as is consistent with rigidity. (3) By compelling the volume of air to spread out into thin sheets so that all of it while passing through a radiator should be kept as nearly as possible in contact with the enclosing metal; this is accomplished by having the conduit through each section of the radiator, 1×7 ins., rectangular in form, instead of round or square. The method of piping consists in having the area of the conducting pipe equal to the areas of the conduit or conduits in the radiator or radiators served by it; also in having the volume of air necessary to serve a radiator, whether it have one or more sections, conveyed to the radiator in a single round or square column. The conducting pipes of a single section are round 3-in. pipes having a circumference of 9 ins. and an area of 7 sq. ins., while the conduit through the

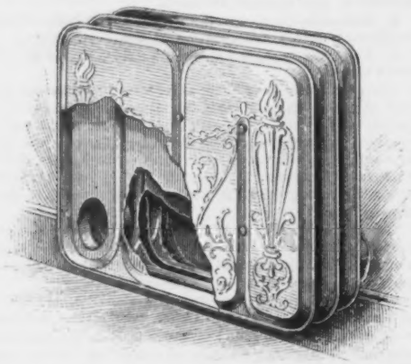


Fig. 1. Sketch of Hot-Air Radiator.

section has a circumference of 16 ins. and an area of 7 sq. ins., or the same area as its supply pipe. A four-section radiator is served by a 6-in. round conducting pipe, having a circumference of 18 ins. and an area of 25 sq. ins., while the sum of the circumferences of the four conduits is 64 ins., and the sum of their areas is 28 sq. ins., or the same as their 6-in. conducting pipes. The extremely flat shape of the conduit through a radiator also prevents any part of the air it contains getting more than half an inch from the enclosing metal, while the center of the air current through a 3-in. round pipe is 1½ ins. away from the containing metal. (4) By facilitating convection, by which method, in order to heat a room, all the air in it must be repeatedly brought into contact with the outside surface of the metal in the radiator. This is accomplished by the flat form of the sections, and where there is more than one section in a radiator, placing them side by side with nothing to obstruct the flow of air between them from below upwards, as heat is imparted to it from the hot air within the sections.

The results obtained in the following experiments by the use of these radiators will be a surprise to those who have not studied the subject. In Feb., 1902, I fitted up a house that was in process of erection, with a discarded wood furnace and rough hand-made radiators. The house was isolated and consisted of a cellar, ground floor, first floor and attic, and contained on the ground floor and first floor, 18,000 cu. ft. of space. The house is now brick-clad, but at that time was neither lathed, plastered nor bricked, and had many openings, large and small, through the outside walls from the cellar to the attic as well as through the roof. Ten radiators were placed in position, nine on the ground and first floor and one in the attic. On the 12th, the outside temperature was 22° F. On the 13th it was 12° F., with moderate wind from the northwest. On each day all the radiators became warm within 45 minutes after the fire was lighted. Two weeks later many of the openings had been covered with building paper preparatory to bricking the walls, the attic and cellar had been roughly shut off from the two floors, the house had been lathed with wet and frozen lath, but not plastered, the temperature was 18° F., with moderate west wind. The ground floor was brought to 63° F. within two hours after the fire was lighted. These results are phenomenal when we remember the wet lath and the enormous difficulty of heating a building when the plaster is wet. The radiator in the attic was 20 ft. horizontally and 20 ft. perpendicularly distant from the furnace; it was the last of five on its main—two others having been supplied from the same pipe on each of the lower floors. The air entered this radiator at a temperature of 213° F., and left it at 122° F. Some of the heating experts present at this test believed, or at least said, while admitting the results, that the system would not stand a practical test of the many turns in the piping needed to place radiators at the points needed or preferred by house owners, nor would it succeed with the slower fire, as from anthracite coal.

To meet these objections, and having in the meantime had the radiators made in a presentable form, I fitted up my own house with them in 1903. The house was an old-fashioned frame of two stories, the part served with radiators containing 17,000 cu. ft. divided into ten rooms and two halls. There being no basement nor cellar, the furnace was set upon the ground floor and burned anthracite coal. It had never been repaired nor had its location been changed, although it had been in use for 13 seasons. The ten radiators installed were supplied from three main pipes. Two of these pipes conveyed the air through the attic before it reached any of the seven radiators; they served the ground and first floors. It was an overhead system, as shown in Fig. 2.

The tinsmiths finished their work and a fire was lighted at 2 p. m. on the 19th of February. The next morning at 8 o'clock the temperature outdoors was 6° F., and in the house it varied from 44° to 60°. The house was never as cold again, and after the first few days the system was better balanced and the variation within the house seldom exceeded 10° and only once reached 16°: when the temperature outdoors was about 11° with a gale blowing from the northwest, the temperature within varied from 54° to 70°.

In passing from the furnace to two of the radiators, the hot air travels a distance of 62 and 71 ft., passing through the attic to do so. In making the circuit from the furnace to these radiators and back to the furnace the air travels 107 ft. and makes 13 turns (exclusive of the radiators, which if counted would make the air travel 116 ft. with 21 turns), showing conclusively that turns in the piping have little or no effect upon the flow of air when it is confined and used in this way, and demonstrating how easily any probable conditions in any medium-sized building can be met with this system.

The foregoing results are much better than could be obtained with this furnace under the old system, for before the installation of the radiators it had been found necessary, in order to get the same comfort, to supplement the furnace with two stoves and a large drum on the first floor. The latter utilized the heat from the smoke pipe of the furnace and it and the two stoves were done away with after the radiators were put in. Over and above this a very large percentage of the heat generated by the furnace is wasted, while the hot air is passing through the pipes in the attic before reaching the radiators. For the large amount of piping in the attic kept it at a higher temperature, notwithstanding attempts at insulation, than either of the lower floors, which of course was not the case before the radiators were used.

The diameter of the fire pot in this instance is 18 ins., because the builders of the furnace claimed that that size pot was plenty large enough to heat 18,000 cu. ft. But this, as was not unusual in those days, was an overestimate, and at the present time it is unusual to find a reliable hot-air furnace maker making such a claim even when the familiar instructions—"Locate the furnace so that all hot-air pipes shall be as short as possible" and "Place registers as close as possible to the furnace" are closely followed. In this test of hot-air radiators these rules have been purposely violated, and the departure from them is so wide that it is believed that 50% of the heat from the furnace is lost in transmission to and through the attic. It is not anticipated that any one in his senses will adopt such an overhead system except under compulsion, neither can it be expected that the results under the conditions of this test should be ideal, yet they are the basis of the confidence felt that the proper installation of these hot-air radiators will demonstrate beyond all question, during the coming winter, a decided economy in the consumption of fuel through their use.

A MUNICIPAL ASPHALT PLANT RECOMMENDED FOR NEW YORK CITY.

The subject of repairing asphalt pavements has just been taken up by a grand jury in New York city. The jury stated that after personal inspection and the examination of witnesses it had found that the pavements were in serious need of prompt repairs, and that radical changes should be made in the repair methods now in vogue. The jury also stated that:

> The same contractors who eagerly seek to procure, and promptly proceed to execute, contracts for new work, are dilatory and more or less evasive, when called upon to do repair jobs under the "maintenance" obligations of existing contracts.

With the increase in the area of asphalt the grand jury believes that the evils now existing will increase, unless there is a change in methods. It then discusses adversely the maintenance clause in the present contracts, and advocates a municipal asphalt plant for repairs as follows:

We hold that a good business doctrine is enunciated by the statement when a manufacturer or contractor, as a seller, produces and delivers up to the standard of his agreement, in full compliance with specification set forth, that his obligation should cease with such delivery, and that, with usual business logic, it becomes the duty of the purchaser to maintain according to the varying conditions of wear and tear.

With direct reference to an asphalt contract, it seems to us clear that no seller can fully foresee the strain or wear that may be put upon a street, under varying conditions liable to rise, and that, if he is compelled to guard himself against such contingencies, the buyer must expect to find the compulsory precaution hidden somewhere in the selling price.

The information furnished to the grand jury by Borough President Cantor and Commissioner Livingston shows clearly that the reduction by them of 15-year "maintenance terms" to those of five years, has had the effect of aiding the securing of great reduction in the price of asphalt contracts, and yet, as has been shown, serious delays and difficulties intervene in many ways when companies, with new work before them, are called upon to perform the more tedious and annoying labor of repairs, oftentimes at scattered localities from a geographical standpoint.

It therefore seems to us quite clear that an abnormal reversal of proper sequence is perpetrated, when complaint as to a broken pavement reaches a public official, is noted by him, and forwarded to a contractor to receive attention, largely at the latter's pleasure or opportunity, a filtration, so to speak, through two channels, with unquestioned delay.

We recommend as a needed improvement of both method and practice, that the official should be equipped to repair the break at once, thus minimizing the delay in bringing about response to a public need.

The functions of the municipal asphalt repair plant should be, in our opinion, rigidly confined to the character of work indicated by its title, and thus the legitimate employment of private capital along the lines of plant instalment and initial production would in no way be endangered or assailed.

In conclusion, the grand jury urged that the officials in charge of the streets should hold the contractors rigidly to their agreements, and that

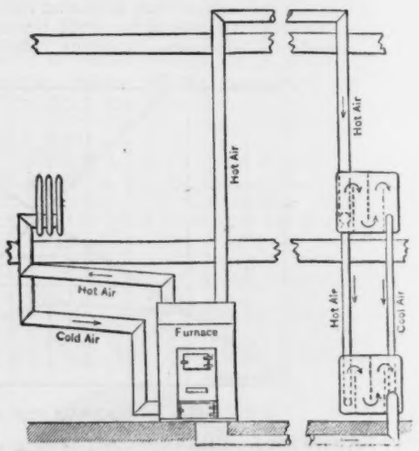


Fig. 2. Piping of Two-Story Dwelling for Hot-Air Radiators.

the contractors "are in honor bound" to respond to calls made upon them for repairs, "thereby proving and establishing their business good faith with the city."

IMPORTATIONS OF IRON AND STEEL into the United States during the fiscal year 1903 were larger than in any preceding year since 1891, and with that single exception are larger than at any time within the last 20 years. The following table compiled by the Bureau of Statistics of the Department of Commerce and Labor shows the imports of iron and steel by articles for the year ending June 30, 1903:

Iron ore	$ 2,351,278
Pig iron	16,830,502
Scrap, for remanufacture only	2,019,258
Bar iron	1,844,486
Railway bars	2,874,188
Hopp, band, or scroll iron or steel	93,864
Steel ingots, blooms, slabs, billets and bars	9,834,236
Sheet, plate, and taggers' iron or steel	481,481
Tin and terne plates and taggers' tin	3,209,915
Wire rods	1,137,370
Wire, and wire articles	643,848
Manufactured iron and steel:	
Anvils	26,405
Chains	66,039
Cutlery	1,782,970
Files, file blanks, rasps, etc	82,485
Firearms	728,077
Machinery	4,085,825
Needles, hand-sewing and darning	442,887
Shotgun barrels	232,544
All other	5,200,812
Total, excluding ore	$51,617,312